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OF THE

AMERICAN WATER WORKS ASSOCIATION

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VOL. 6 JUNE, 1919 No. 2

COMMENTS

EXPERIENCES IN COLLECTING BILLS

Many water-works superintendents advocate legislation making charges for water used a lien on the property. From the viewpoint of the water-works operator such legislation is doubtless desirable. Occasionally charges arise entirely beyond the scope and power of legal procedure.

An instance of this kind recently occurred in Philadelphia, as an accompaniment of one of the rapidly increasing cases of damage to a fire hydrant by a recklessly driven automobile. To the credit of the Police Department a report of the accident with the number of the automobile license was turned in to the Bureau of Water, and a bill in the amount of approximately $60 was rendered to the registered owner of the vehicle. No check being forthcoming, a search disclosed the fact that the owner of the automobile was with the United States Army in France in the service of the Government, and still further investigation brought to light the fact that the soldier, when entering the Government service, had laid up his car for storage in a garage. The garage owner was then investigated and was able to satisfy the Bureau that he had no knowledge of the car having been taken out on the street. Further investigation, however, developed the fact that an employee of the garage, entirely unauthorized, had been out on a joy-ride with the automobile, during which he had collided with the fire hydrant. Search for the

employee brought to light the fact that he was dead. The Water Bureau then demanded settlement of the claim from the executor, and this individual responded with the advice to "go to the devil." The whole transaction was then referred to the City Solicitor who returned the claim with the comment that hell and the devil were out of his jurisdiction.

CARLETON E. DAVIS.

THE AUTO TRUCK PERIL FOR THE POST HYDRANT

A new problem has appeared for the water-works engineers and superintendents in the destruction of post hydrants by the heavy auto trucks which are now so generally used throughout the country.

Hydrants are so located with reference to the curb line that there is no danger of their being damaged by a horse-drawn truck, but the substitution of auto trucks for horse-drawn trucks has created an entirely different situation. In the winter time, when the street is covered with snow or ice, especially the latter, an auto truck skidding in turning the corner will slide up over the curb and strike a hydrant with sufficient force to break it off, usually at or about the ground line. At all times of the year an auto truck is liable to back up to the curb without paying much attention to the overhang, and on account of the weight and power in the truck, will break off a hydrant, and the driver may hardly be conscious of having struck any obstacle. If a hydrant is placed in a vault, such as is frequently the case with the high-pressure hydrants in the Borough of Manhattan, City of New York, the hydrant branch may be ruptured in place of the hydrant itself. Such an occurrence was the cause of the flooding of the basement of the Postal Telegraph Building, where many thousands of dollars of damage resulted.

The experience in New York has indicated that the strength of the hydrant cannot be made sufficient to prove an effective answer to the problem. High-pressure hydrants, which are $10\frac{7}{16}$ inches in diameter and have a thickness of metal of $\frac{15}{16}$ inch, have been broken as well as the much lighter low-pressure or ordinary type of hydrant.

There are several possible answers to be made to the problem presented. Some will advocate the flush hydrant, which, to date, has not made appreciable progress in this country. In Boston, where both flush and post hydrants are used, the troubles resulting

from the breaking of post hydrants by auto trucks have caused a change to be made in the type of post hydrant, which up to date has opened with the pressure. The compression type hydrant opening against the pressure is to be substituted, and it is expected that the use of the flush hydrants will not be extended. Moving the hydrants from the curb line back to the building line is another possibility. The continuance of hydrants in their present location and adding to the cost of maintenance the replacement of hydrant stand pipes when broken, is the answer which will probably be adopted by the majority of water-works superintendents. This cost is materially reduced if the stand pipes are repaired by welding, which can be done successfully with modern machinery at a cost of from $6 to $8 per stand pipe.

The subject is one of such general interest, and offers such opportunity for fruitful discussion, that a prominent place is to be assigned to it at the Round Table portion of the program for the coming convention at Buffalo. The outcome of such discussion should be very helpful to the water works fraternity.

W. W. Brush.

LAYING WATER MAINS IN ALLEYS

The plan of laying water mains, gas mains, sewers, steam mains, telephone conduits and all other underground structures, used for the service of the public, in the alleys instead of in the streets is frequently advocated and sometimes tried.

The principal reason given by advocates of the plan is that by laying mains in alleys the streets will be opened less frequently and consequently pavements will be less often disturbed. This is a very good reason, for we will all agree that it is seemingly impossible to replace pavements so as to avoid bumps and holes. It can be done by skillful and conscientious workmen, in the case of first-class pavements, although the poorer the class of pavement the more difficult the job of replacement. The laying of conduits in alleys for light, power, telephone and telegraph lines, is frequently adopted and seems to be practicable in certain districts. Sanitary sewers might often be laid in alleys, but where combined or storm-water sewers are required the difficulty increases because catch-basins must be provided at street intersections.

Of course it is true that nowadays alleys are paved as well as

streets, and that there is as much difficulty about replacing a pavement in an alley as in a street, but alleys are not used for fast driving so much as are streets and the scars made by excavations would not be so prominent in alleys as in streets. The principal objection to the alley is that it is not wide enough to accommodate all the underground work and that alleys, as usually laid out, are not continuous and usually run in one direction only. Gas mains and water mains, but especially water mains, must be laid as a gridiron, crossing frequently at right angles, and this is seldom possible if alleys are used exclusively.

Water mains supply fire hydrants which are necessary and are usually and properly located at street intersections. As an example: The commercial district of Davenport is covered by a perfect gridiron of good-size mains. All the alleys in this district run east and west. If the mains were laid in these alleys the north and south mains would have to be laid in streets. In such case hydrants at street intersections would be on one main only while alley hydrants, which are less numerous, would be near intersecting mains. Hydrants are also required half way between street intersections on east and west streets as well as north and south streets, and if mains were laid in alleys these hydrants would have to be eliminated in some cities as there would be no space in which to set them in 20-foot alleys.

A few years ago the writer tried to lay a small water main in an alley because it was desirable to avoid cutting a new pavement. There were several conduits in this alley used by telephone, telegraph and electric power companies and at the entrance of the alley were three manholes of considerable size. The water pipe could not be laid between the manholes because they overlapped. It could not be laid through them because the companies owning them objected and it could not be laid under them because the ground was solid rock and blasting would disturb the conduit system. The new pavement was cut and the pipe was not laid in the alley.

Gas and water services require shut-off cocks and curb boxes for outside control, and these are best located on sidewalks if they are to be accessible when needed. Curb boxes should always be located on public property. When located in alleys heavy vehicles are liable to pass over and break them. It is not desirable to place a curb box at any point where it may be covered up by

ash barrels, manure boxes, or other material. When locating a meter in a commercial district the front entrance to a building is superior to the rear entrance in most cases.

In existing residential districts the same objection above mentioned regarding the width of alleys and that they usually run in one direction only, and not continuously, applies. In new additions provision for laying mains in alleys could be made so long as one policy prevailed, but then mains running at right angles to the prevailing direction of the alleys would have to be laid in streets. When blocks are 500 to 800 feet between cross streets sufficient hydrants would not be possible for proper protection, conveniently located, when mains were in alleys.

In two cases the plan of laying water mains in alleys has been tried in new additions in Davenport. In one case, a high-class residence district, private rights-of-way had to be provided for hydrant branches and for some mains, in order to complete the layout. One eight-inch main in this district now lies very deep under a brick wall and other expensive improvements in the yards of fine houses. The alley was vacated in this case. Parts of the district were platted without alleys and mains were laid in the streets, necessarily. In another district, houses were built facing, and on each side of, a wide boulevard. The water mains were laid in the alleys behind each row of houses. This plan worked all right until houses were built on the far side of the next parallel street when mains had to be provided for them. There being no alley, the main was laid in the street and we now have parallel mains one half block apart.

In one Iowa city the principal main was laid through the alleys in the commercial district. Fire hydrants located at street intersections were supplied from this main by long branches of smaller pipe. These long hydrant branches added to the expense of the system, added to the loss of head between the main and the hydrant, and there being no flow in the pipe between the main and the hydrant the danger of freezing would naturally be greater than in a normal installation.

The writer is willing to concede that the alleys could be utilized for some utilities, probably wiring and sanitary sewers, but contends that water mains, for reasons applying to safety and economy, had best be laid in the streets.

C. R. HENDERSON.

THE WATER SUPPLY BUSINESS

The water supply business, particularly when conducted by a city, is subject to a peculiar difficulty. Just as well established custom encourages, although ethics discourages, the practice of dissimulation in equine exchange, to speak scientifically, so custom and the love of adventure make it an illegal but joyous sport, to housewives and to captains of industry, to cheat—no, to get the better of—those whose business it is to furnish water. It is really not good form to pit one's wits against the inspector and meter of a private company, in the opinion of some who find pleasure in this gentle line of graft. The company is a private affair, and while taking its water surreptitiously is not like robbing a hen roost, yet it has the taint of kinship to burglary, in the minds of a few who would gladly enjoy this pastime but are held back by conscientious or sentimental reasons, just as they hesitate at attendance at a cock fight while yearning to be among those present. But when it comes to the quiet intercepting of a needful supply of water from the mains of a municipal water department, few there are who do not feel that being water takers and residents makes them part owners in the plant and, ipso facto, free to take what they desire. To their minds the rules and regulations, the schedules of charges, the meter reader and the bill collector are doubtless needed, such things give a business-like tone to the municipal government, but how many persons actually think of them as they think of the price lists and clerks in a department store?

Some years ago a manufacturing company completed a plant in a large city. It spent considerable time in a careful search for a man to take charge of its mechanical equipment. Finally one was selected; his employers for many years back were interviewed and all united in praising his intelligence, willingness and honesty. In the early days of his service he tuned up the equipment and then started in to introduce economies. Every fortnight or so he reported something of this sort, and finally told the president that only one really important thing remained to be done. This was to cut a hole through the basement wall of the office building, tunnel out to the street water main and tap on a private connection so that the water bills could be cut down. The president is a good man, he had been superintendent of one of the biggest bible classes in the country, but he acknowledged in telling about the incident

that for a moment he was tempted. However, he braced himself against the strain and firmly declined. The feature of the incident that is important is this: The superintendent, the guaranteed honest man, was taken aback and urged the plan, not only as an established custom but also as perfectly justified by the large sums that would be paid for water taken through the regular metered services, even though more than half the total quantity required was water percolating through officially unknown channels, what might possibly be termed a large subterranean supply from a free aquifer, to speak scientifically once more.

This half forgotten incident was recalled by a report sent to the JOURNAL by W. H. Dittoe, chief engineer of the State Department of Health of Ohio. Complaint had been made about the unsatisfactory quality of the water supplied by the city of Newark, in that state. Part of it was drawn from filter units in the bed of the Licking River and part was raw river water; all of it was chlorinated. The investigation showed that a better supply was needed and that the consumption was extraordinarily high. The superintendent and the director of public service were greatly puzzled, for practically every service was metered and there was no clue to the source of waste. Finally a pitometer survey was made which disclosed these facts: The pumps had been furnishing a daily average of 3,932,000 gallons, of which 552,000 gallons of daily leakage were detected and stopped, and 895,000 gallons were taken by a private company through an unauthorized connection to a fire protection line which entered its premises from a 12-inch city main. The company claimed that it had absolutely no knowledge of this connection, through which it was receiving every day almost as much water as was registered by the meters with which every recorded service had been equipped. As a result of the publicity given this case there was a little later another great reduction in the consumption of water, explained locally by the possibility that some other industrial corporation may have discovered that it was using water without due authority and had hurriedly removed all traces of this sad deviation from the narrow way of rectitude by an over-zealous employee, whose acts would never, of course, be countenanced by a responsible officer.

The incident has another real human side. For some years the consumption had been around 3,000,000 gallons daily and rose to a peak of 4,100,000 gallons a year ago. When the pitometer sur-

vey was over, the pumps were called upon to deliver only 1,845,000 gallons. The city felt morally certain that the unrecorded draft of 895,000 gallons per day through the fire protection line had been going on a long time, but it decided to ask payment for only the water taken at that rate since the beginning of the year. The bill amounted to the tidy sum of $49,000. The company which had been taking the water was not pleased at this, and offered to pay $6,700, which was the value of the water actually proved to have been taken through this connection about which it denied all knowledge. The city threatened to cut off all water and the company obtained an injunction against this, and is now defending a suit for the $49,000 in one of the higher courts of the state.

Incidents like these show that a municipal water department must have an extra-sensitive managment. It must recognize that a good many people go after city water with a jimmy and dark lantern; they do not look on it as a serious lapse from the path of righteousness to break into a city water main. Public sentiment is not aroused when they are caught; people merely laugh at their predicament when this capture is made. But if the size of the works is to be kept within reason, and the cost of an ample supply of wholesome water is to be maintained within bounds, this wholesale theft of water must be stopped. It is one of the thankless tasks of the water works manager, a troublesome and annoying task, for it makes him suspect many prominent citizens and look questioningly at many large industries. But human nature being what it is, the task is a very real one if the department is to be run on business principles. No business man turns his back when thieves are about.

JOHN M. GOODELL.

THE TRENTON, N. J., FILTRATION PLANT

By F. W. Daggett

For a great many years Trenton has taken its supply of water from the Delaware River. The territory above the city, while not as densely populated as that below, still has many large towns and small cities, all of which discharge their sewage directly into the river. In very few cases is this sewage given any preliminary treatment. At times of flood, the Lehigh, the Delaware's largest tributary above Trenton, brings down great quantities of fine coal dirt from the anthracite washeries along its banks. At such times the Delaware at Trenton, 50 miles below the mouth of the Lehigh, is often the color of writing ink. At other times it is the color of coffee due to wash of the clay banks immediately above the city.

With these facts in mind, one is not surprised that for at least twenty-five years various projects had been proposed for improving the water supply. Most of them recommended filtration of the river water. Examinations of the sub-surface conditions surrounding Trenton had proven that very little could be expected in developing an adequate supply from artesian wells.

This view was supported by Mr. Allen Hazen, in 1900, in his very complete report on the purification of Trenton's water supply. Mr. Hazen recommended filtration of the river water, and suggested five different projects. Three of them used the slow sand method of filtration, while the other two were rapid sand or mechanical types of plants. Mr. Hazen favored one of the slow sand projects. Owing to its high initial cost, and also to the fact that this project proposed to convert the newly completed distribution reservoir into a sedimentation basin, Mr. Hazen's recommendation was not received favorably by the water board.

For the next ten years very little was done. In 1910 typhoid fever was very prevalent in the city, the polluted water supply being undoubtedly the principal source. In 1911 the city adopted government by a commission, and abolished the old political water board. The water department then came under the direction of

Mr. J. Ridgway Fell, Commissioner of Streets and Public Improvements. Mr. Fell had formerly been city engineer and realized that something else than talk was needed to conserve the health of the city. He put the matter in the hands of Mr. George A. Johnson, consulting sanitary engineer of New York. Mr. Johnson, recommended that the city build immediately a sterilizing plant to disinfect the water by the use of hypochlorite of lime, to be followed a few years later by rapid sand filtration. This suggestion was adopted, and within six months after the commission government took hold, the sterilizing plant was put in operation. The deaths from typhoid in the year following decreased over one-third. This decrease has steadily progressed ever since, and in 1916, with complete filtration as well as sterilization, the deaths from typhoid are only 15 per cent of what they were in 1911.

Work on the filtration plant was started in January, 1913, and it was put in operation in November, 1914. It is located on the river bank about 300 feet upstream from the pumping station where both the high and low lift pumps are placed. There are two of the latter, each with a capacity of 20,000,000 gallons daily. They are of the centrifugal type driven by 220-volt direct-current motors. A constant water level is maintained in the sedimentation basins by means of a float which controls the field of these motors and thereby their speed. The electric power is generated by two 200-kilowatt DeLaval turbo-generators, one of which is always in reserve.

The filtration plant was designed to handle 30,000,000 gallons per day. At the time the plant was being built the average daily consumption of the city was over 20,000,000 gallons. Since then, owing to pitometer surveys and to an increase in the number of metered services, the consumption has been reduced to an average of about 17,000,000 gallons per day.

There are two sedimentation basins, each with a capacity of 2,000,000 gallons. Either one or both may be used, thereby varying the period of subsidence to suit conditions. The water on entering, after being treated with a coagulant, is evenly distributed across one end of the basin by means of weirs and baffles. This prevents eddies and cross currents, and gives a minimum entrance velocity to the water, which secures very favorable conditions for the rapid subsidence of the suspended matter. Owing to restricted area, the basins were made 20 feet deep. It was expected that they

would not be as effective as a larger and shallower basin, but experience has shown them to be able, with the proper alum dosing, to remove from 60 to 80 per cent of the turbidity.

As the basins cover less ground space than is usual for basins of their capacity, the depth of mud accumulated is quite great. After a year's run, it is often 6 feet deep at the inlet end, tapering off to about half that at the outlet. There is also a marked difference in its consistency. As one would expect, the material at the inlet end is heavy and coarse, with a large amount of sand, leaves and such material. As the floors of the basins are at about the elevation of the normal river level, all the sludge has to be pumped out. In each basin there is a pump designed to handle about 15 per cent of mud or sand when mixed with 85 per cent of water. If these figures are exceeded and the pump becomes clogged, the direction of rotation of the impeller is reversed and a water jet opened into the casing. These means rarely fail to clear the impeller of all obstructions except leaves, small twigs, sticks and the like. Such things must be kept out of the pump unless they are made small enough to pass through. To accomplish this, the pump was enclosed by a fence 2 feet high of $\frac{3}{4}$-inch copper screen. A fire hose is played on this screen, which is forcible enough to tear most of the leaves into small pieces, which can readily pass through both the screens and pumps.

The sixteen filter units are arranged in two rows of eight, and each has a normal capacity of 1,875,000 gallons per day, and a sand area of 652 square feet. The combined air and wash-water system is used. The strainer system is all cast iron pipes with brass strainers. The laterals are embedded in cement level with their tops. The strainers have a deflecting plate on the top which causes a downward flow of the wash water against the floor of the filter. This was expected to prevent any organic growths on the floor of the beds, and after four years operation none has been observed. The strainers have the trap tube feature common to many systems of this type, and which has proven most effective in ensuring a uniform air distribution.

The rate of washing is about 13 gallons per square foot per minute. This is about as high as it is possible to go with this style of filter bottom unless either larger gravel were used and more of it, or some means taken to prevent the gravel from being lifted. The whole washing system is very efficient, and the wash water used has averaged

about 1.5 per cent. During 1918 it was 1.3 per cent. The author cannot recall any figures from the plants using high rate wash without air that are better than the above. Four years experience at Trenton seems to show that the air wash is really a wash water saver, and also has other advantages in aerating and stratifying the filtering medium.

The filter gravel is a foot deep and ranges in size from $\frac{3}{4}$ to $\frac{1}{8}$ inch. The filter sand had a depth of 30 inches when placed, but about 2 to 3 inches has been lost in washing. Three months after the plant was started, this sand had an average effective size of 0.45 mm. and a uniformity coefficient of 1.40. Two years later an analysis showed the average effective size to have increased to 0.53 mm. while the uniformity coefficient had remained practically constant at 1.38.

Sulphate of alumina is used as a coagulant, the average consumption being a ton a day. During high water stages there is a deficiency of alkalinity in the river water and soda ash is required to obtain proper coagulation. When in 1916 the price of alum went soaring, this plant was one of the first to install apparatus for making its own alum. The Hoover process, which was perfected at the Columbus filtration plant, is used here. As this process had not been developed when the plant was built, it had to be fitted in afterward as best it could, and the work has been handicapped in this respect. On this account the making of alum at the filtration plant has lately been abandoned, and a plant is now being erected which will have a railroad siding and other facilities for doing the work more economically. Acid will be purchased in tank cars, and possibly pulverizing machinery installed so that the less expensive grades of southern bauxite can be utilized.

In 1916, when commercial alum was selling for about $60 a ton, the Hoover process showed a saving of about $25 a ton. In 1917 and 1918 as the price of commercial alum dropped slowly, the saving was reduced to a minimum of about $8 a ton. Then the manufacture was discontinued for the reasons just stated. With the benefit of three years' experience, it is expected that the new plant will show a saving of from $10 to $15 on a ton, depending on the relative costs of commercial alum and sulphuric acid. Wholesale manufacturing methods, such as outlined herein, are found to be most effective in balancing increasing costs of labor, and thus keeping the total cost of operation at a minimum. It is found that the

Hoover alum gives results equally as good as the standard alums, but its higher percentage of insoluble material (12 to 15 per cent) requires minor changes in the chemical devices.

Outside the removal of turbidity, which occasionally gets as high as 2000 parts per million, the plant must cope with a water which is highly polluted with sewage bacteria at all times. Soon after filtration was started, liquid chlorine was substituted for bleach as a sterilizing agent. From $1\frac{1}{2}$ to 2 pounds are added per million gallons of water. Samples of the effluent are tested for B. coli every four hours, and all tests are confirmed. Great care is taken to eliminate these bacteria, and duplicate chlorinators are always ready in case of trouble with the one in use. In 1918 the total number of B. coli tests made were 2267. Of this number only nine-tenths of 1 per cent were positive in 1 cc. and 2.8 per cent were positive in 10 cc.

During the winter of 1917–1918 great difficulty was had in getting an adequate supply of chlorine, owing mostly to transportation troubles. The cost of this material also seemed to the officials to be too great. It was decided to try out a method of making chlorine electrolytically from salt. Upon investigation by the author, it was found that there were only two filtration plants making their own chlorine; one at the Little Falls plant of the Montclair Water Company, used the Nelson cells, and the other at the plant of the Montreal Water and Power Company used the Allen-Moore cells. The latter was very completely described in a paper by F. H. Pitcher and J. O. Meadows, printed in the JOURNAL, volume 4, page 337. The Trenton plant is so similar that no detailed description will be given here. Both installations use the Allen-Moore cells, designed by the Electron Chemical Company, of Portland, Maine. A new 300 ampere cell was designed for Trenton, which is half the amperage of the Montreal cells. The four cells have a maximum output of 75 pounds of chlorine a day. The usual output is about half of this, and only two or three cells are in use most of the time. Each cell requires three and a third volts. The current is taken from the water works own 220-volt direct-current service, and reduced to the proper voltage by a motor-generator set at the filtration plant.

The amount of chlorine produced depends on, first, the amount of brine flowing through the cells, and second, the amperage. At Trenton the former is kept as nearly constant as possible, while the

latter is varied to suit the pumpage. This method of control does not give as good economy of production as a constant current method would, but it is more accurate, and overdoses of chlorine, with a possible obnoxious taste, are avoided.

The plant was placed in operation in November, 1918, and is producing chlorine at a cost of 4.2 cents a pound exclusive of depreciation and investment charges. At that time the city was paying 17 cents a pound for its liquid chlorine. It takes about 4¼ pounds of salt and 1.6 kilowatt hours of electric current to make a pound of chlorine gas. At Trenton, as at Montreal, the effluent from the cells, consisting of undecomposed salt and caustic soda, is wasted, as the quantity produced does not justify its reclamation. In a larger plant it would pay to reclaim them in a marketable form; the salt to be used over again, and the caustic either sold or used in place of soda ash or lime. Such a plant could produce chlorine at very low cost. It would probably be very little more than the cost of the electricity required.

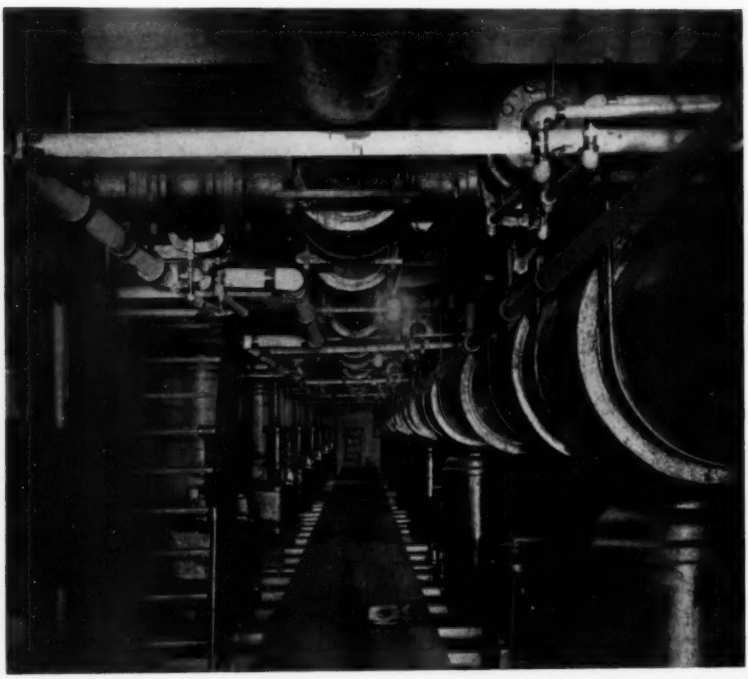
Table showing results of operation of the Trenton filtration plant for the years 1915 to 1918 inclusive

	1915	1916	1917	1918
Total amount of water filtered (M. G.)...	6276	5906	6081	6775
Average water filtered per day (M. G.)...	17.19	16.15	16.69	18.58
Average filter runs (hours).....	35	42	48	43
Average percentage of wash water.....	1.8	1.4	1.6	1.3
Average alum used (lbs. per M. G.).....	93	87	112	100
Average chlorine used (lbs. per M. G.)...	1.7	1.5	1.8	1.7
Average turbidity raw water.....	57	18	30	29
Average turbidity filtered water.....	0	0	0	0
Average color raw water.....	28	25	27	28
Average color filtered water.....	6	10	5	8
Average alkalinity raw water.....	26	25	24	23
Average alkalinity filtered water.....	20	21	19	19
B. coli; percentage of positive tests in 10 cc. filtered water	3.2	5.5	7.5	2.8

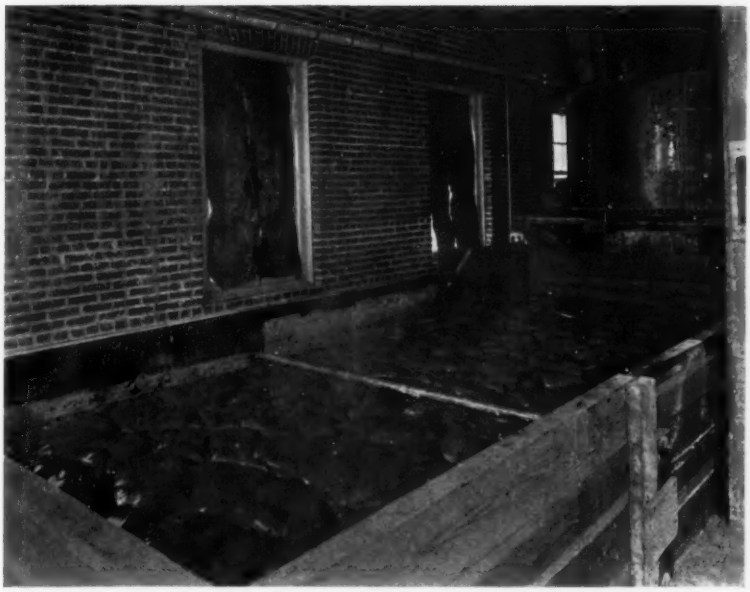
For eight years past Trenton has been sterilizing its water with chlorine, using three different methods: First, bleach; second, liquid chlorine; and last, electrolytic chlorine gas produced as required. Each step has marked a distinct improvement and in the last one the water officials feel that they have the most economical and efficient sterilizing equipment known at the present time.

GENERAL VIEW OF THE WATER FILTRATION PLANT AT TRENTON, N. J.

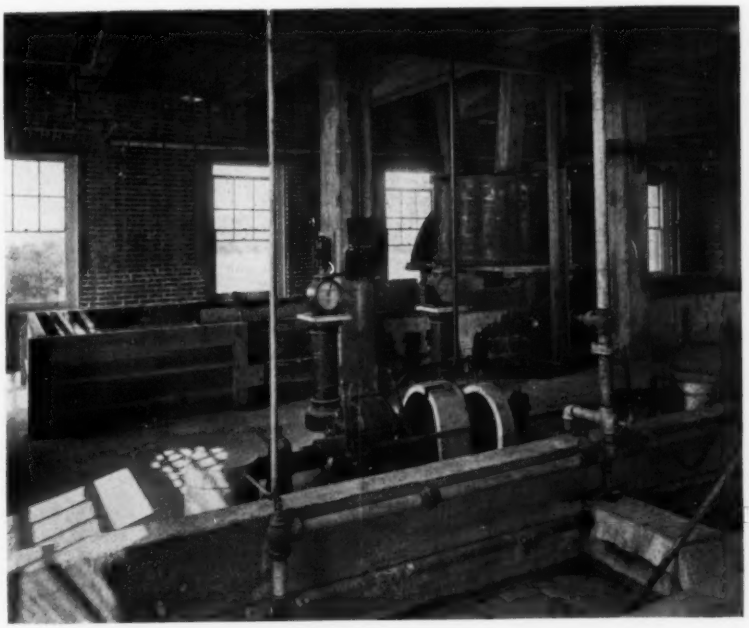
INTERIOR OF THE FILTER HOUSE



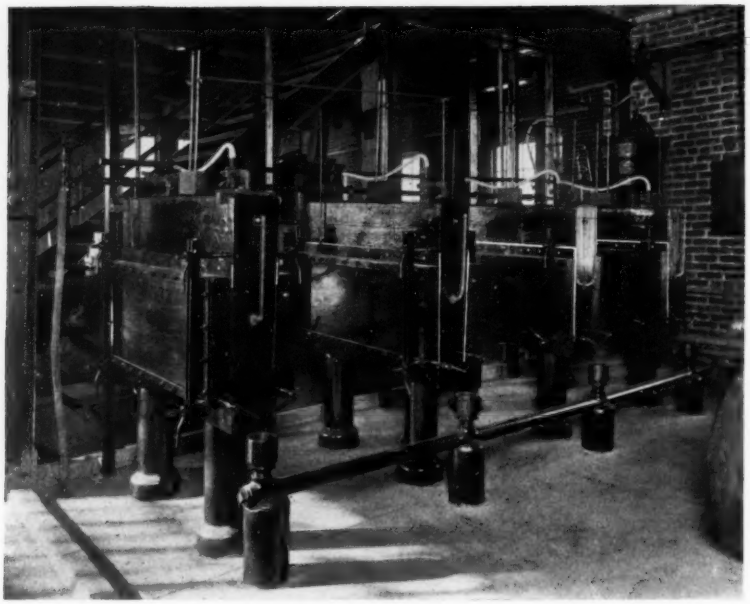
THE PIPE GALLERY



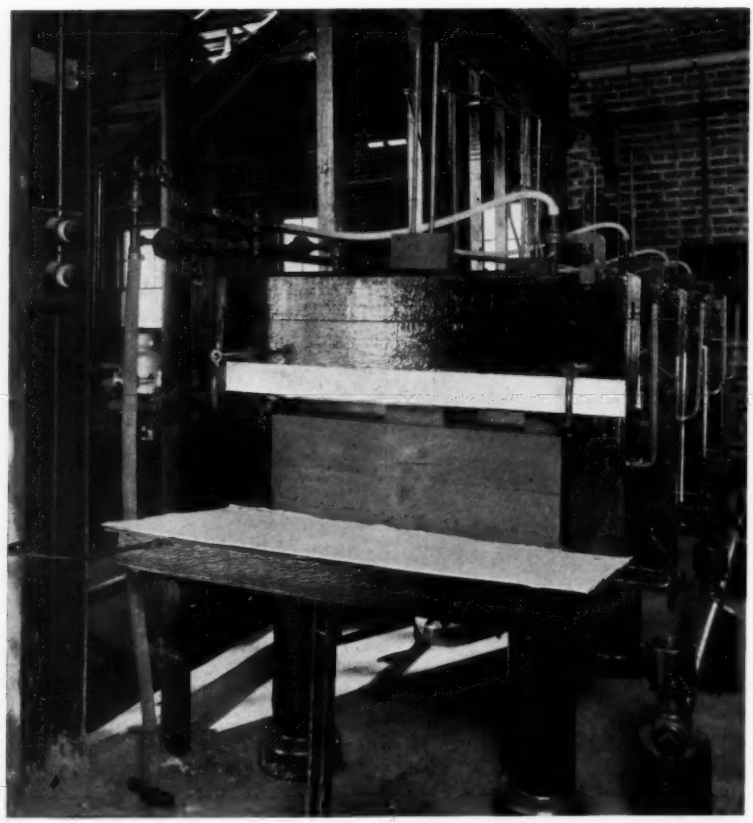
Alum Boiling in the Crystallizing Box



Apparatus for Making and Dissolving Alum



General View of Cells, Electrolytic Chlorine Apparatus



Interior of a Cell, Electrolytic Chlorine Apparatus

SETTLING BASIN EFFICIENCY AT PHILADELPHIA

By Joseph S. V. Siddons

The Torresdale filter plant, located 10 miles above the center of the City of Philadelphia on the Delaware River, includes a series of slow sand units (final filters) which total 48¾ acres in area and 120 preliminary or roughing beds each having an area of 1200 square feet. A covered filtered water basin of 50,000,000 gallons capacity receives the filtered and treated water prior to its flow into a tunnel leading to the Lardner's Point Station, from which the filter delivery is pumped into the city distribution system.

Prior to October, 1916, water was introduced to the preliminary units by pumping directly from the river. Following this date the suction to the station was transferred to a sedimentation basin which had been constructed along the river shore adjacent to the slow sand filter location. Excavation was made by dredging, and cribbing was sunk and loaded with gravel and earth to form a dike which acts as a dividing wall between the river proper and basin. The intake is provided with control gates and baffles and over it is built a head house where chemicals may be stored and treatment applied. The basin is approximately 2000 feet long and 500 feet wide and ranges, due to tidal conditions, from 15 to 21 feet in depth. The quantity of water within the basin limits permits of a subsidence period of approximately twelve hours.

The introduction of the sedimentation basin reduces to a marked extent the amount of suspended matter which at times of a turbid river would add to the removal load of the filter. Should conditions warrant, suction from the river may be effected by opening the gates of the former intake. The water enters the sedimentation basin about one-half mile upstream from the old location, thereby offering less chance of flood tide pollution from the city's drainage.

In February, 1918, during a period of high turbidity, a coagulant was applied. The effect of the basin upon the plant operation on dates subsequent to the basin's use, is given in table 1.

TABLE 1

Million gallons yield of Torresdale filters between cleanings

	MAXIMUM		MINIMUM		DAYS RUN		
	Per filter	Per acre	Per filter	Per acre	Maximum	Minimum	Average
To the end of 1916	569	758	30	37	148	11	30
During 1917	815	1087	53	71	240	25	99
During 1918	695	927			233		

The above are yields following washing by the Nichols' method. The record for 1918 is not complete, as a number of filters started in the last quarter have not up to date completed their runs.

The number of cleaning operations on the final filters for years 1915 to 1918 are respectively 719, 491, 360 and 312. Due consideration must be given to the labor conditions which prevailed during 1918, when shortage necessitated a revision of filter cleaning practice and the effort was to return a bed to service as rapidly as might be permitted. The use of the basin made it practicable to handle smaller quantities of sand and give a serviceable filter unit which would furnish a satisfactory effluent. The amount of sand handled during 1917 and 1918 was respectively 157,000 and 90,000 cubic yards.

The effect of the subsidence period in the basin is shown in table 2, which represents turbidity counts taken over a six months' period. The removal by the preliminary units is also expressed.

TABLE 2

Turbidity counts during six months

	MAXIMUM	MINIMUM	AVERAGE	PERCENTAGE REMOVED
River	34	6.0	11+	*
Applied to pre-filters	31	4.0	10+	14.4
Applied to final filters	8	0.5	1+	93.0

* Average is carried to decimals to produce per cent removed.

The Delaware water normally permits of an alum dose of 1.5 grains per gallon and this coagulant is applied in solution. Typical results expressing turbidities are shown in table 3.

TABLE 3

River and effluent turbidities

SULPHATE OF ALUMINA APPLIED	RIVER	APPLIED TO ROUGHING FILTERS	APPLIED TO FINAL FILTERS
1.6 grains per gallon	176	57	19
1.5 grains per gallon	150	60	16
1.7 and 1.3 grains per gallon*	75	46	8

* One day of this period the readings were 100, 20 and 2, which reflects favorably upon the action of the pre-filters and arouses the thought that this series of beds might be operated as a mechanical plant.

Results of cleaning operations during 1917 are given in table 4.

TABLE 4

Yield in million gallons, Torresdale filters, 1917

METHOD	NUMBER OF FILTERS	TOTAL	PER FILTER			PER ACRE		DAYS RUN		
			Maximum	Minimum	Average	Maximum	Minimum	Maximum	Minimum	Average
Nichols wash	146	41,781	815	53	287	393	4.0	240	25	99
Raked (once)	133	15,820	304	29	119	159	4.3	96	13	37
Raked (twice)	24	1,316	100	19	55	73	3.8	35	8	19
Brooklyn	45	6,196	292	42	138	184	3.9	96	20	47
Scraped and piled	12	1,474	136	24	123	164	3.8	50	11	43
Totals	360	66,587			185	247	3.9			63

An approximate yield and the cleaning operation which produced the runs are given in table 5.

TABLE 5

Cleaning operations and following yields

NUMBER OF BEDS		YIELD
Washed	Raked*	
1	35	Less than 50,000,000 gallons
8	46	Between 50,000,000 and 100,000,000 gallons
32	63	Between 100,000,000 and 200,000,000 gallons
47	12	Between 200,000,000 and 300,000,000 gallons
28	1	Between 300,000,000 and 400,000,000 gallons
21	0	Between 400,000,000 and 500,000,000 gallons
6	0	Between 500,000,000 and 600,000,000 gallons
1	0	Between 600,000,000 and 700,000,000 gallons
1	0	Between 700,000,000 and 800,000,000 gallons
1	0	Maximum 815,000,000 gallons
146	157	

* Maximum of rakes 304,000,000 gallons.

SOME OBSERVATIONS AND EXPERIENCES IN THE OPERATION OF COAGULATING BASINS

By James W. Armstrong

The two coagulating basins of the Montebello filters of the Baltimore water works are each 317 feet long, 232 feet wide and have an average depth of water of about 15 feet. A central baffle wall extends about three-quarters of the length of each basin. Water is admitted through five sluice gates, spaced at equal intervals, and after passing around the baffle, is withdrawn through a similar number of gates. The water passes over a baffle at the entrance and over a skimming wier at the exit. It was the hope of the designer that the water would move with reasonable uniformity through the basins, but this has not been realized.

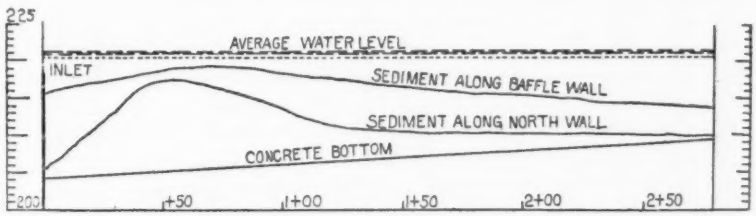
In the course of operation it was noticed that the movement of water through each of the two coagulating basins was different, although they were operated in parallel and apparently under the same conditions. It was also noticed that the water, while following characteristic lines in each basin, seemed to vary from time to time.

In order to learn something more definite regarding the exact movement of the water and its effect on the efficiency of the basins, a series of sixteen different current measurements was made. For making the observation, six identical aluminum floats were used. Each float was cross-shaped in plan and was composed of four sheets of aluminum each 6 by 12 inches, inserted in a pine strip. The floats were just heavy enough to half submerge a large cork, to which a number was pinned. The portions of the floats exposed to the wind were very small, and as the observations were not made on windy days, it is believed that their movements indicate the currents through the basin with reasonable accuracy. Most of the observations were made with the floats submerged just below the surface or 4 feet below.

The floats were started at the entrance of the basin and readings were taken, locating their position, every six minutes. The floats in the main channel, both surface and at the 4 foot depth, generally

moved forward at relatively high velocities, while those along the outer edges of the basin moved in very erratic ways, sometimes getting caught in eddies in the corners and staying there for hours. Basin 1 usually showed an eddy at the entrance in the northwest corner, probably due in part to a slight difference in the alignment of the baffle. Basin 2 showed no eddies at the entrance, but both basins showed eddies in the far corners and usually along the outlet side of the baffle wall.

On August 14, 1917, surface floats were used for making current measurements in basin 1. The velocity of the floats varied considerably. Some of them moved *forward* at rates as high as 32 feet per minute. On the outlet side of the baffle, two of them were carried *backward*, adjacent to the baffle for three-quarters of its length. On August 18, 1917, the basin was cleaned and on August



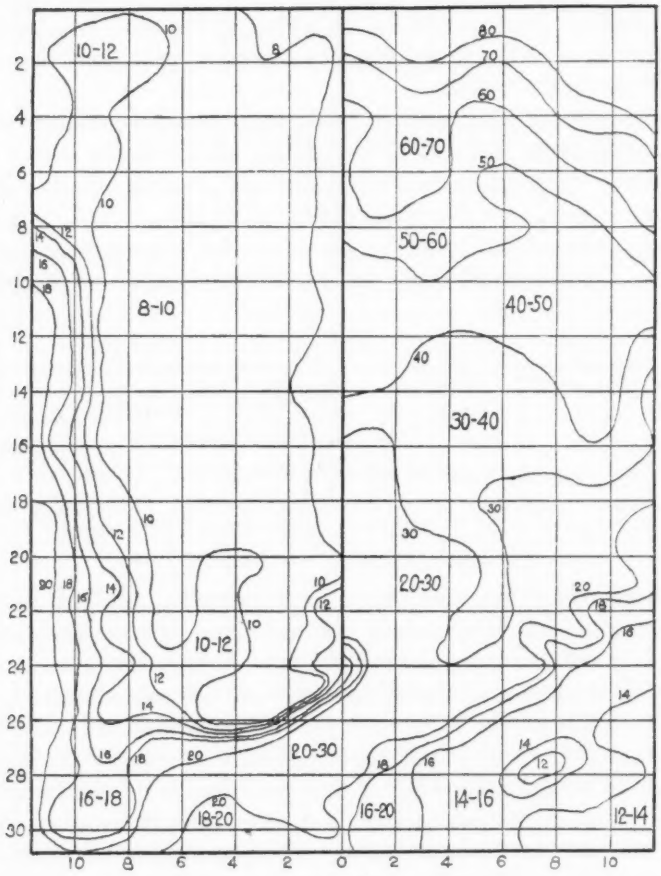
SEDIMENT IN BASIN 1 ON MAY 25, 1916

21 other surface float measurements were made. On this occasion the floats moved along entirely different lines. One of them circled around in the northwest corner, the other four moved slowly along the baffle wall at velocities of from 2 to 12 feet per minute, and on the outlet side of the baffle showed no decided forward movement.

The greater velocity and more uniform movement of floats on August 14 was due to the fact that mud had filled the lower part of the basin, necessitating higher surface velocities. Most of the float measurements showed that higher velocities were obtained near the baffle wall on the incoming side and the outer wall on the outgoing side. A survey of the mud surface just before cleaning showed that the heaviest deposits followed approximately the lines of maximum velocity of flow, and that the greatest depth of deposit was at a point a little over 50 feet from the entrance. The corners where eddies occurred had the least depth of mud.

On August 16, 1917, two days before cleaning the basin, a turbidity survey of basin 1 was made. Readings were taken at

intervals of 10 feet, with a United States Geological Survey turbidity rod. The heaviest turbidities were observed at the entrance, and from there they decreased along pretty well defined but irregular lines toward the rear of the first half of the basin. In



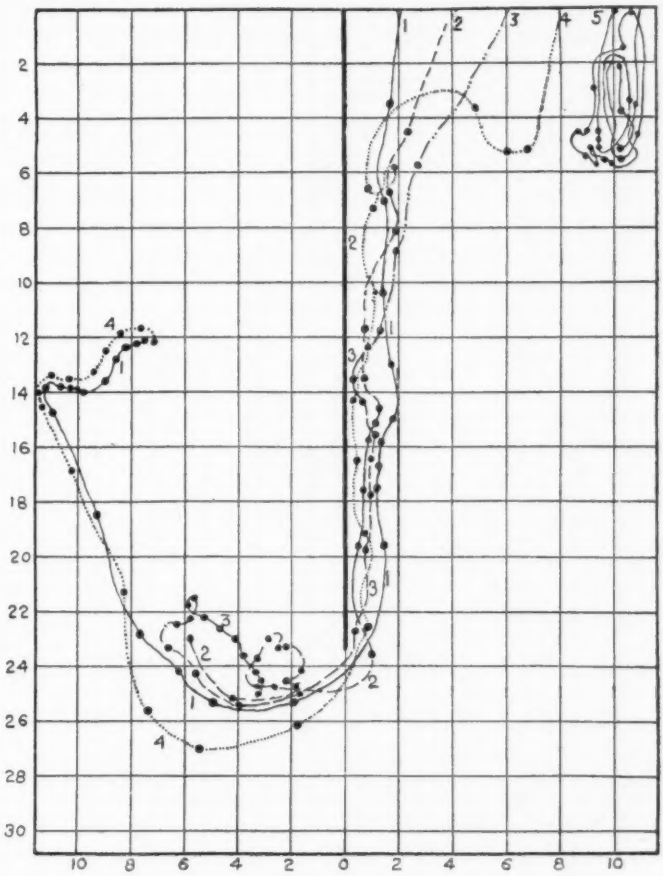
SURFACE TURBIDITY MEASUREMENTS, BASIN 1

Readings taken with U. S. G. S. turbidity rod; Venturi rate of flow 115 m.g.d.; August 16, 1917.

the second half of the basin the maximum turbidities followed approximately the lines of greatest velocity. This test seemed to indicate that the velocity at the entrance was too high to secure

the best subsiding value and that uniform velocities are necessary for uniform subsidence.

At one time, for a short period only, the bacterial counts were slightly higher in the effluent from the coagulating basins than in

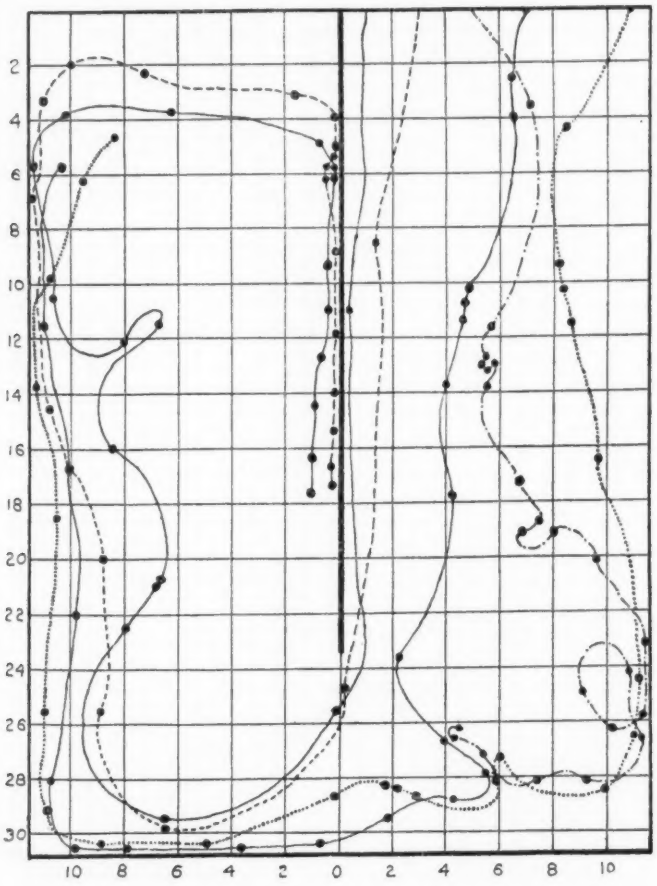


CURRENT MEASUREMENTS WITH SURFACE FLOATS, BASIN 1

Readings of floats taken every five minutes; Venturi rate of flow 123 m.g.d. August 21, 1917.

the raw water. The increase of growth in the basins led to the belief that bacteria were multiplying in the places where the currents were sluggish and were being gradually swept into the main current and carried to the filters.

In order to test the accuracy of this theory, several careful bacterial surveys were made. Numerous samples were taken at points showing the greatest difference of turbidity and velocity. One set was taken near the surface and another at a depth slightly above



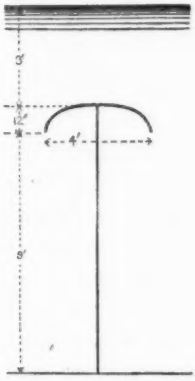
CURRENT MEASUREMENTS WITH SURFACE FLOATS, BASIN 2

Reading of floats taken every five minutes except first reading on each float which had six minutes interval; Venturi rate of flow 128 m.g.d.; August 14, 1917.

the mud line. The samples taken near the entrance of the basin showed slightly higher counts than those taken at other places, but none of the results showed any evidence of increased growth in any part of the basin.

In addition to the variations in velocity and the lack of uniformity in mud deposits, other irregularities were noticed in the functioning of the basins. As an illustration, it was noted on one occasion that basin 1 gave decidedly better results than basin 2, but upon making a slight change in the opening of one of the inlet gates, the condition was shortly reversed and basin 2 gave markedly better results than basin 1.

This phenomenon raised the question as to the possible effect of undercurrents upon the subsiding value of the basins. It was argued that if the elements of water in moving through the basin could be kept in nearly parallel lines and if the movement could be confined to the surface of the water, leaving the lower stratum undisturbed, much greater subsiding value could be obtained.



WOODEN BAFFLE ACROSS BASIN 1

With the purposes of testing out this theory, some modifications were made in the existing entrance baffle, and an additional cross baffle was built in basin 1. In order to give a positive outward sweep to the water entering the basin, increase the surface velocity, and prevent returning undercurrents, a wooden apron about 2 feet wide and sloping in an upward direction was attached to the top of the existing entrance baffle. At the end of the existing concrete baffle, about 240 feet from the entrance, an additional wooden baffle was built entirely across the basin. No changes were made at the outlet skimming wier. The new baffle was 10 feet high and was capped with a top 4 feet wide, which projected 2 feet on each side. The top was curved downward at the ends in the form of a

parabola. Water usually flowed about 3 feet deep over the top of the baffle.

The following reasoning was responsible for the building of the unusual type of cross baffle.

The reduction of the cross sectional area would tend to establish a more uniform flow across the top of the basin and create a quiet zone near the bottom that would permit better sedimentation. It was well known that when flowing water meets an obstruction such as a baffle, there is a decided tendency to pick up sediment and carry it over the top, and it was argued from the known characteristic of a parabola to reflect rays of light in parallel lines, that the sediment in the rising currents of water, would, upon striking the parabola, be thrown downward and deposited, instead of being carried over into the next compartment. The curved top over the rear of the baffle is only useful in gradually reducing the velocity of water flowing over the top.

After this baffle had been in service for a number of months, the basin was cleaned. It was found that the apron extending across the top of the baffle at the entrance of the basin had been built a little too near the surface, and the velocity of entering water was so great that it carried most of the sediment outward as far as the cross baffle. The results at the cross baffle, however, were most gratifying. The parabolic top had evidently acted exactly as had been anticipated, for the mud was deposited adjacent to the baffle, almost to its full height, and sloped gradually backward to the entrance of the basin. The baffle on the return side had not performed so well, probably owing to the fact that the water in passing over it was not flowing in a direction normal to its face. It is believed that if this baffle had been placed about 70 feet nearer the outlet, it would have worked better.

As soon as the opportunity is afforded the apron controlling the incoming water will be lowered so as to decrease the velocity at the entrance, and the baffle on the outgoing side will be removed forward. It is hoped that then the sediment will be deposited more uniformly.

This hopeful but imperfect experiment is made public with the thought that it may stimulate others to make further experiments or lead them to publish information which they already have.

THE TYPHOID FEVER EPIDEMIC AT XENIA, OHIO

By F. H. Waring

Xenia, Ohio, had a population of 8706 at the time of the 1910 census; of this population 2052 were negroes. The water comes from two sources. The older supply is pumped from a plant called the Springfield Pike station, which furnishes a mixture of surface and ground water. This has been shown to be of poor quality on several occasions, and on this account a hypochlorite disinfecting equipment was installed in 1914 to treat it. The second source is a group of drilled wells at the so-called Cincinnati Pike station, which furnishes water that has always been regarded as good from a sanitary viewpoint. As a rule about two-thirds of the total supply is furnished by the drilled wells, but during dry weather this source does not yield as much water as normally and at such times the quantities of water from the two stations become more nearly equal.

There were only two cases of typhoid fever in Xenia in 1917 and there were none in 1918 up to August 27. A case was reported on that day and with the exception of August 28 and September 17, new cases occurred every day until September 18, when the last one was reported. There were forty-four cases in all, quite generally distributed throughout the city in proportion to the density of the population. All were within the district supplied with water and the only case in a home without a connection with the street mains was that of a woman who used the public water supply where she worked. The city water supply is used by most of the residents of the city, and there are very few wells in service.

The milk used by the persons taken sick came from six dealers, from groceries and restaurants, and in five cases from cows kept by the patients. Two cases used only condensed milk. The ice cream that had been used by five-eighths of the cases came from many different sources and the same was true of the fresh vegetables used by those who were taken sick. No case had eaten shell fish within the incubation period. Only eleven cases resided in homes where other cases occurred and there were eight cases where no typhoid fever

occurred in their vicinity before or after these cases. There were two families with three cases each where contact may have spread the disease. There had been no public gatherings which could be regarded as contributing to the spread of the disease. From these facts it seems clear that the common source of the infection was the water supply.

The epidemic was investigated in September by representatives of the State Department of Health. They found that the disinfection of the Springfield Pike supply was carried on by dissolving dry bleach and adding the solution to the water at a predetermined rate, according to the variation in the rate of pumping. The work was carefully done and the bleach solution had been added in definite quantities and with regularity for two and a half years before the epidemic. Analyses of the water as it was drawn from taps in the city showed that it was of unsatisfactory quality so an analysis of the hypochlorite solution was made to ascertain if there was anything the matter with that. It was found to have only about one-fifth of the proper strength, which further investigation showed to be due to the use of inferior bleaching powder, for the method of using the powder and the quantity employed were correct provided the quality of the powder was normal. Since this substance was guaranteed to be of proper strength, it follows that the undertreatment of the city water was purely accidental, but it is significant that such a possibility always exists when disinfection of a public water supply is necessary to render it safe for domestic use. The State Health Department recommended the abandonment of the Springfield Pike station and the substitution of a source of supply above criticism, and, as a temporary insurance against a recurrence of the trouble, the use of liquid chlorine instead of hypochlorite of lime.

The introduction of a liquid chlorine equipment was made at once. The water is examined twice a week at least by C. P. Hoover, chemist in charge of the Columbus water purification plant, and the results are reported regularly to the State Health Department. Following this investigation at Xenia, every water disinfecting plant in Ohio has been placed on the same basis, and a reputable chemist has assumed control of the chlorination at each plant and reports regularly to the State Health Department each month.

THE OPERATION OF CANTONMENT WATER-WORKS

By Lieut.-Col. G. A. Johnson

The water-works at the military camps and cantonments are operated under the direction of Col. C. D. Hartman and Lieut.-Col. Geo. A. Johnson, of the Maintenance and Repair Branch of the Construction Division of the Army, commanded by Brig.-Gen. R. C. Marshall, Jr. On August 1, 1918, the Maintenance and Repair Branch issued a series of instructions for the operation and maintenance of the water and sewerage works at these military cities, which has been supplemented from time to time. There is a great dearth of general information on this subject in print. Accordingly permission has been received for the publication in this place of so much of the orders as relate to the management of the water works:

Plans. 1. The Officer in Charge of Utilities should have on file for reference a complete set of plans of the water and sewerage works connected with the camp. Plans of water works owned by municipalities or private water companies which supply the camp should also be on file at the utilities office.

The plans should include:

Source of water supply,
Water purification plant,
Pumping stations and force mains,
Storage tanks,
Water distribution system,
Sewer system,
Sewage disposal works.

2. These plans should be kept up to date together with all necessary supplementary information.

Records. 1. Records of the operation of all features of the water and sewerage works must be kept in detail. The Officer in Charge and each operator should be provided with a diary in which to record all facts and observations. The diaries should show the number of workmen employed on different branches of the work, orders

for supplies and equipment and receipt of same and any unusual conditions as well as regular operating data. All important observations should be recorded, such as odors, flies, fly breeding, unsightly conditions about the plant and any other condition which needs remedying.

2. Complete records should also be kept of all special investigations such as waste of water, leakage from water distribution systems and from plumbing fixtures; tests of pumps and other equipment; and determination of the efficiency of the different parts of the water and sewerage works.

3. All field notes having an important bearing are to be transcribed in regular or special reports made to this office.

Records of population, water consumption and pressure. 1. In addition to the records of operation of the different features of the water works, the following general records are to be kept daily and transmitted to this office once a month.

a. Population, Military and Civilian.

b. Number of Animals.

c. Water Consumption. Total for the day and maximum and minimum rates during the day.

d. Pressure. Maximum, minimum and average at selected high and low points in camp.

GROUND WATER SOURCES

1. Rate of well pumping. Wherever several wells are available water should not be taken at abnormally high rates from any one well, as so doing is liable to result in clogging of the strainers with fine sand. So far as practicable the wells should be operated at a moderate rate and the peak loads taken from suction or storage wells.

2. Observation wells. Two-inch observation wells should be driven at suitable locations in the well field to determine the fluctuations in ground water table.

3. Water depth in wells. The depth in wells where air-lifts are used should be determined weekly to ascertain the submergence of the air nozzle. In some cases this can be done by inserting a pipe of small diameter between the casing and the drop pipe. In case the annular space is insufficient for this a 2-inch observation well should be driven immediately adjacent to the well.

4. Test of pumping equipment. Each well should be tested from time to time to determine the capacity and the efficiency of the pumping equipment. If meters can readily be installed at each well this is the best method. Where this is not feasible, other ways can be found such as measurement of the accumulation in suction or storage wells.

5. Relief of clogged strainers. Where it is found that a well is not delivering the expected volume of water it is an indication that the strainer is partially clogged with sand. Such sand can usually be removed by reversing the flow, under as high a head as can be obtained. In some cases it will be advisable to pump water from other wells, by means of centrifugal or other pumps, through the drop pipe to wash the sand out into the annular space between the drop pipe and casing. In other cases it will be necessary to dismount the pump, remove the drop pipe and bucket the sand out. It is much more economical to keep present wells in first class condition than to be continually driving new ones.

6. Surface flooding of wells in low ground. Where wells are located in low ground liable to be flooded, the tops of the wells should be raised to an elevation above possible high water and the pump house and machinery should be protected against flooding if this is liable to occur.

7. Chlorination of flooded wells. If wells, or the low ground about the wells, should become flooded with polluted water, the wells must not be operated until arrangments have been made to chlorinate the water until all danger is past. This condition is particularly dangerous in the case of shallow wells. Even in the case of deep wells there is danger of the polluted water finding its way into the supply around the outside of the casing or between the casing and the drop pipe.

8. Guard well fields, tanks and pumping stations. The well-field, particularly in the case of unsealed wells, suction or storage tank and pumping stations, should be carefully guarded to protect the supply from accidental or wilful contamination. The actions of men working at or near the source of supply should be carefully watched.

9. Chemical and bacterial tests. Chemical and bacterial tests of the water should be made daily, or with sufficient frequency to show the character of the water at all times. Where treatment is required, such treatment must be subject to laboratory control.

Where chlorination or filtration is required, tests must be made at least daily of the raw water, treated water, and tap water.

10. Daily records. Daily records are to be kept of the quantity of water pumped, the operation of the pumps, power used, repairs made, work done on wells and all other operating facts, observations, and tests.

SURFACE WATER SOURCES

1. Rainfall and run-off data. In the case of surface water supplies it will be important to secure rainfall and run-off data, particularly in the case of limited supplies, to determine the available quantity and the adequacy of the supply.

2. Watershed policed. Even though the water supply be treated with chlorine or filtered, a reasonable amount of policing of the watershed is necessary to prevent undue pollution. Reservoirs, dams, gate-houses, etc., must be properly guarded.

3. Intake racks and screens kept clear. Racks and screens at the intake must be kept clear. Racks can generally be taken care of by rakes or squeegees, but screens should be removed alternately and occasionally scrubbed. Protection against excessive clogging of racks and screens can be secured by the use of a floating boom arranged like a scum board several feet, in front of the rack or screen.

4. Analytical control. Chemical, bacterial and microscopical analyses of the raw, treated and tap water should be made daily, or with sufficient frequency to show the character and quality of the water at all times. Wherever chlorination or filtration plants are provided, they must be operated under chemical and bacterial control.

5. Algae growths in reservoirs. Objectionable growths such as algae may produce offensive tastes or odors in surface waters, particularly when stored in large reservoirs. Copper sulphate properly applied is effective in checking or reducing such conditions. Explicit instructions will always be furnished by the Washington office before this treatment shall be put into effect.

6. Records. Records are to be kept every day of all operations, repairs, measurements, observations and tests, as well as the results of the laboratory tests.

CHLORINATION PLANT

1. Instructions for operating chlorinators. Manufacturers' instructions for operating chlorination apparatus should always be available to the operator who should become thoroughly familiar with its mechanism and competent to adjust and repair it if necessary.

2. Preliminary precautions. In order to prevent possible clogging of the regulating valve at the top of the chlorine cylinder with oil or other impurities, this should be taken into the open air and the valve turned sufficiently to blow off a little gas. When doing this care should be taken to stand on the windward side and avoid inhaling the gas.

3. Weighing the chlorine. The liquid chlorine cylinders should be set on platform scales and the automatic apparatus checked up frequently to assure proper dosing.

4. Reserve supply of chlorine. An ample supply of the sterilizing agent should be kept on hand to tide over any delays in shipment or in transit, and empty cylinders must be returned promptly.

5. Duplicate apparatus. When used constantly, duplicate apparatus should be available and in proper working order, and all necessary repair parts should be kept on hand.

6. Analytical control. This treatment should be subject to laboratory control, and bacterial tests should be made with sufficient frequency to ensure adequate and economical use of the sterilizing agents.

7. Records. Records are to be kept daily of the volume of water treated, the rate of application of the chemical, bacterial results on the raw water, treated and tap water, and all other operating facts and observations.

FILTRATION PLANT

1. General operators. No attempt will here be made to explain all of the operations incident to the coagulation and filtration of water, water softening or iron removal. An experienced operator is to have supervision of each plant, whether owned by the government or by a municipality or by a private company. Furthermore, when any doubt arises regarding any detail of operation or results the Washington office must be notified promptly, and advice will be immediately forthcoming.

2. Improvements and enlargements. Where any part of the plant is found to be incapable of doing its work properly so as to insure a water supply adequate in quantity and satisfactory in quality, notation as to the character of the deficiency and apparent possible remedy should be transmitted to this office, where such recommendations will be given due consideration and such improvements or enlargements, as the case may require, will be authorized.

3. Records. Among the records which are to be kept at the filtration plant are the following:

a. Turbidity, color, alkalinity and bacterial content of raw water.

b. Kinds and amounts of chemicals used.

c. Number of filters in service and length of runs.

d. Quantity of water filtered.

e. Filter units washed and quantity of wash water used, time of application of water and of air.

f. Total bacteria and B. coli in raw, filtered and tap water.

g. Labor employed.

h. All other operating facts and observations.

PUMPING STATIONS

1. Isolation of fire risks. The general instructions which have already been issued with respect to keeping fire-risks from proximity to pumping stations, wooden storage tanks, etc., should be rigidly enforced. In case the fire hazard at a pumping station is especially serious the type of construction known as "slow burning" should be provided.

2. Guard pumping stations. Pumping Stations should be under guard at all times.

3. Pump operating troubles. It is not feasible to go into all of the operating troubles of the various types of pumps used at water works, but some of the principal troubles are discussed below.

a. Deep well pumps. With deep well pumps, troubles are experienced from sand or other substances getting between the ball or conical valve and the valve seat, the wearing out of the sucker rods from binding on the side in the case of crooked wells, and the wearing out of bearings, particularly on unbalanced, single-acting pumps. Sand can generally be removed by operating the pumps at high speed aided in some cases by quickly increasing and decreasing the pressure. If this be not successful, it will be neces-

sary to remove the working barrel and sucker rods. This should be done occasionally in any event, and provision should be made in the well house for its easy accomplishment. The wearing out of sucker rods caused by rubbing on the side of the well casing frequently can not be corrected, and if the difficulty is serious either through continual replacement of rods or power loss it may be more economical to discontinue the well.

b. Impeller pumps. Impeller pump troubles are due largely to wearing out of bearings and the wearing of casings by runners because of improper adjustment or the wearing of alignment bearings. Oiling, also, particularly with the American well type, sometimes gets out of order. In any case the only remedy is the replacement of the worn parts.

c. Centrifugal pumps. Centrifugal pumps are generally reliable, and get out of order mostly because of poor adjustment of the runner due to the balancing devices not taking proper care of the thrust and allowing the runner to rub on the casing. The proper adjustment of the runners within the casing is very important.

d. Air lifts. Air lift devices are extremely simple but require much care in adjustment of air piping and nozzle to the size of well, depth of water and quantity to be pumped. These matters must be taken care of by a man experienced in this class of work, and frequent observations of water level must be made to secure proper adjustment.

4. Repair shop. A well-equipped repair shop or workroom with a competent machinist should be available at every important pumping station in order to maintain the machinery in proper working order. Duplicate parts should always be kept on hand.

5. Machinery replacements. It is likely that because of the impossibility of obtaining machinery exactly fitted in all cases to the requirements, some of the machinery installed might economically be replaced by more efficient apparatus. Efficiency tests will determine the facts, and any request for replacements should be accompanied by the results of such tests.

6. Use oil engines daily. Where gasoline or oil engines are used for auxiliary power, they should be tested daily. The following instructions shall be posted in each pumping station in which there is an oil, gasoline or kerosene unit:

Daily tests of at least ten minutes duration shall be made of all fire pumps operated by oil, gasoline and kerosene engines. Dur-

ing tests pumps shall be operated at rated speed and full fire pressure maintained.

Motor should be started slowly and note made whether there is any trouble in this operation.

Motor should then be brought up to speed and note made of any uneven explosions, unusual noises, etc.

Records shall be made of (a) duration of test; (b) pressure maintained; (c) remarks as to defects if any, and (d) name of operator.

Under "Remarks" shall be noted:

Defect in ignition, starting, oiling system and gas supply.

Amount of gasoline on hand.

Are cocks on gasoline supply line open?

Condition of lubricating oil supply.

Whether oil feed is working properly.

Date batteres last recharged.

Whether gate valves on suction and discharge pipe are in good working order.

7. Layout maps. In each pumping station there should be hung on the wall a blueprint showing all pipe connections, valves, pump locations, etc., in or about the station. Whenever changes in piping are made the map should be revised to show such changes.

8. Records. The daily record should show the rate of pumping, pressure, units in service, repairs made, power used, amount of fuel on hand and amount used for different purposes, and all other operating facts and observations.

FIRE HYDRANTS

1. Paint fire hydrants. Fire hydrants should be made as conspicuous as possible by painting bright red.

2. Inspection of hydrants. Hydrants should be inspected frequently to see that they are in good working order with the caps in place and not screwed on too tight.

3. Protection of hydrants against injury. Any hydrants so located as to be in danger of injury from traffic or other cause should be adequately protected by guard posts or railings.

4. Pipe branches for fire hydrants. Four (4) inch pipe should be used for hydrant branches only up to thirty (30) inches in length. When branches longer than 30 inches are necessary six (6) inch branches and pipe should be used and the hydrant set on a 6 inch by 4 inch Redwood large end bell.

5. Hydrant number. Each hydrant shall have a number which shall be painted on the barrel. Figures should be at least three inches in height. Location of hydrants by number shall be shown clearly on a map. This map shall be kept up to date, and all additions, changes or repairs shall be reported by number. Copies of this map should be on file at the various offices concerned in the operation of the water works and the matter of fire protection.

STORAGE TANKS

1. Storage tanks covered. Elevated storage tanks and reservoirs should be covered with a substantial roof and all openings for ventilation should be properly screened to prevent the access of birds and insects. Manhole covers should be kept closed and locked.

2. Guard tanks by fencing. Where such tanks or reservoirs are accessible they should be fenced in to keep out all persons not duly authorized on the premises.

3. Fill tanks at times of low draught. Storage tanks or reservoirs should be filled at times of low draught and not by extraordinary pumping at times of maximum consumption; and peak loads should as a rule be taken from storage.

4. Tanks guarded and inspected. Storage tanks and reservoirs should receive frequent inspection, and in the majority of cases should be under guard. Particular attention should be given to wooden tanks liable to be set on fire by railroad locomotives.

5. Automatic recording devices. Height of water gauges with automatic recording devices should be installed so that records of the daily fluctuations in volume of water in storage may be available. Such records are of value particularly in indicating the efficiency of operation and in computing water consumption.

DISTRIBUTION SYSTEM

1. Pressure reduction prohibited. Reduction in pressure on the distribution system by cutting storage tanks out of service, by the use of pressure reducing valves, by partially closing valves or other means, will not be permitted without special authorization from this office. If local conditions appear to demand a departure from this policy a description of conditions is to be submitted for an investigation.

2. Cover over pipe lines. All pipe lines and service pipes should have sufficient earth cover to protect them from freezing. If necessary, pipes should be lowered or additional cover provided. Risers to elevated tanks should be protected with pipe frost proofing if necessary. Sawdust is not suitable because it becomes saturated with moisture and freezes.

3. Inspections. Frequent inspection of the distribution system should be made to maintain all valves in good operating condition, especially those depended upon for use in event of fire.

4. Pipe location maps. A map showing the location of pipes and valves should be accessible to all operators, and all valves which are not standard opening should be shown conspicuous on this map. Any valves temporarily closed to improve service should be so indicated on the map by special push pins.

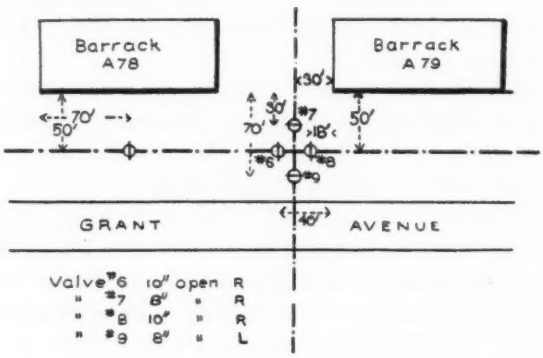


FIG. 1. METHOD OF REFERENCING VALVES

5. Air valve inspection. Air valves along force mains or gravity supply lines should be inspected at least weekly and opened slightly to relieve air in case the valve is stuck.

6. Records. Records should be kept of all observations, repairs, changes, etc., on the distribution system.

7. Gate valve reference book. The location of all gate valves on the distribution system should be determined and shown in a reference book. The usefulness of such a book is obvious. The valves should be located and tied into permanent land marks such as barrack buildings, etc. The measurements from these land marks should be carefully made and entered on a sketch similar to the sample attached. These sketches should be indexed and

bound together to make a reference book. Care should be used in determining whether or not a valve is right or left hand. A valve which opens by turning in the direction the hands on a watch travel, opens to the right and should be indicated "R."

8. Tracings of these sketches should be made and retained as part of the Utilities Records. Blue printed copies on linen should be a part of the Water Works Repair Gang's equipment. A copy should also be on file in Washington.

9. In connection with this Reference Book, a master map of the entire distribution system on a small scale should be prepared, on which valve locations as shown on the sketches would be indexed.

PREVENTION OF LEAKAGE AND WASTE

1. Clothes washing under running water. The washing of clothes under showers or running streams of water results in unnecessary waste. Where feasible, set or movable tubs should be provided for this purpose. In some cases it will prove advantageous to provide a water connection and sink outside the latrine.

2. Overflow of troughs. Water troughs and tanks at Remount Stations should not be permitted to overflow, not only because of unnecessary waste of water, but also because mud about the trough is objectionable.

3. Devices to reduce pressure locally. The use of such devices as perforated lead washers, slugs, and marbles in the faucets and shower sprinkle heads for the purpose of locally reducing excessive pressure and resultant waste, is recommended. Round-headed, self-closing faucets can be advantageously installed where there is a scarcity of water.

4. Regulation to prevent waste of water. Regulations should be issued to prevent unnecessary use of water in latrines, the leaving of faucets and shower heads open after using, etc., and should be rigidly enforced by inspection and policing if necessary.

5. Water waste surveys. Systematic surveys should be made from time to time to locate leaks in water mains and service pipes. Tests for leakage on main lines can be made by pitometer readings, by the use of aquaphones, and by pressure gauges in isolated districts supplied by small pipe lines. Aquaphone tests should be made when the pressure is high, as the sound from leaks will be greater at such times. Localization of waste with master meters can best

be made during the hours of minimum consumption, from midnight to 4 a.m. The flow in sewers may also serve as an indication of water waste or leakage in different localities.

The method of making a water waste survey best adapted to the conditions encountered in camps and cantonments is known as the hose and meter method. The survey may be carried on by this method with the least amount of labor and does not require the services of technically trained men.

Apparatus. The apparatus necessary in making hose and meter tests is: One 2-in. compound type water meter, with fish trap or screen attachment. Pressure gauge attached to hydrant nozzle cap. Aquaphones and fire hose as needed.

Hours for making tests. In order that accurate figures be obtained it is necessary that tests be made when all legitimate con-

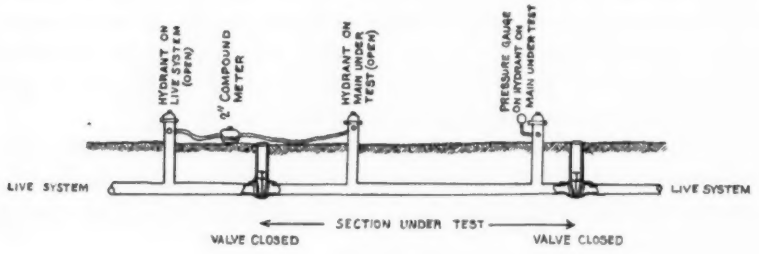


FIG. 2. HOSE-AND-METER TEST FOR LEAKAGE

sumption is at a minimum. For this reason tests must be made at night, preferably between the hours of midnight and 4 a.m.

Isolating the test section. The section to be tested is isolated from the remainder of the system by closing gate valves on the main. These valves should be tested to assure tightness. The accuracy of the test depends on the tightness of the valves isolating the test section. If a leaky valve is encountered it will be necessary to drop back to a valve that is tight.

Setting the meter. The isolated section is connected to the live system by fire hose laid from a hydrant inside the test section to a hydrant outside on the live system. The meter is set on this line of hose. All water consumed by the isolated section must now pass through the fire hose and meter when the hydrants are opened. The meter being of the compound type will automatically and accurately measure both large and small flows.

Hydrants to be flushed. Hydrants should be well flushed before using in order to remove any sticks or stones which may be lodged in the hydrant branches. Care should be taken in joining hose to guard against dirt and stones being admitted to the meter.

Pressure records. A pressure gauge should be attached to a hydrant nearby the testing station in order that record may be kept of the pressure on the section under test. A pressure equivalent to that ordinarily in the system should be maintained throughout the test.

Key map. Before actual tests are begun a map should be prepared showing clearly the division of the distribution system into permanent test sections. Permanent test stations for the indicated sections should be established and given a distinguishing symbol on the map.

Reporting results. Monthly reports are to be made which will describe in detail the progress of the survey. The report covering the first month's work must include the map showing how the distribution system has been sectionalized. A tabulated form of report will be used in conjunction with the written report.

Sub-division tests. When a section is located that demands furthur investigation to localize a flow that is abnormal a sub-division test should be made. That is, the large test section is in turn divided into smaller sections which are tested individually in order that the flow indicated for the large test section may be localized on a short run of main for final analysis and location.

Reports of sub-division tests. Individual tabulated reports should be made of sections in which sub-division tests are made.

6. Night inspections. Faucets, showers, toilets, etc., should be inspected every night, and when found out of order should be repaired or replaced promptly.

7. Records. Complete records should be kept of all observations and investigations relating to water waste and leakage.

ELECTRIC PIPE THAWING

Thawing of water pipes with electric current is now an accepted practice.

An outfit to take care of this work should be a part of the utilities equipment in camps located in that part of the country where

frozen service pipes and mains constitute part of the operating difficulties.

An outfit which contains only materials readily obtained at the least possible expense and the component parts of which may be readily converted to other uses, is to be desired.

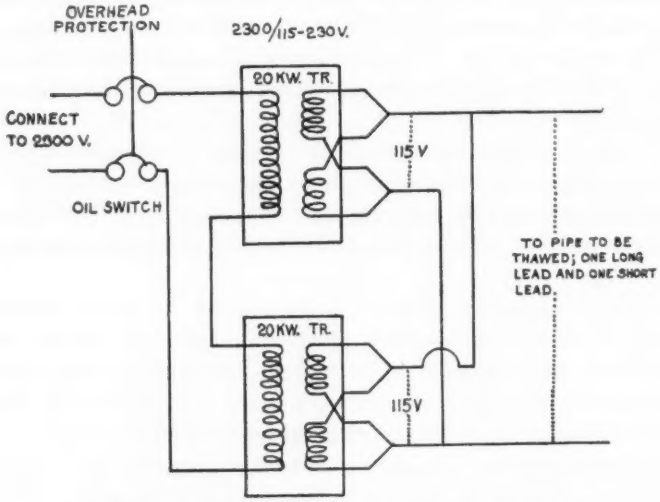


FIG. 3. CONNECTIONS FOR ELECTRIC THAWING

For that reason, the enclosed plan is submitted. The equipment required to make up a thawing outfit according to the plan attached is as follows:

300 feet no. 6 D. B. R. C. Stranded Cable 5000 Volt Insulation.
500 feet no. 00 D. B. R. C. Stranded Cable 500 Volt Insulation.
2-20 KVA 2200/220/110 Volt 60 Cycle distributing transformers.
1-2 Pole S. T. 2500 Volt outdoor type 100 Amp. oil switch with overload trip.

It is thought that most camps will have the majority of these parts in stock. If this is not the case, requisitions should be submitted promptly for such equipment as is necessary to assemble an outfit of this type.

WATER EXPECTANCY IN TUNNELS, MINES AND DEEP WELLS IN HOMOGENEOUS ROCKS

By Robert E. Horton

In planning deep subsurface structures such as tunnels and mines, it is desirable to form some idea in advance as to the amount of water which will probably be encountered and will require removal. Isolated test shafts are costly, and may not show average conditions. It is desirable to supplement such special data by considerations based on general experience as to the yield of underground water under similar conditions.

Data can generally be secured as to the yield of shallow or surface wells overlying the region where the underground structure is to be made. Such data have been published concerning many conditions and kinds of rock as to the average and usual range of yields of surface wells under different conditions.

It is the purpose here to present a method and formula by which data obtained from surface wells can be applied, under suitable conditions, to the estimation of the amount of water which it may reasonably be expected will be encountered in any deep excavation.

The method is limited to cases where the bed rock is somewhat uniform in character throughout all depths from the rock floor down to the bottom of the tunnel or other structure. For example, the method will apply to the estimation of probable water in a tunnel in granite, which extends to the surface, but will not apply to a tunnel in deep-seated granite overlain by thick beds of sedimentary rocks.

Without giving details it may be said that there are physical reasons, well confirmed by statistics and experience, showing that the frequency and water-carrying capacity of fissures, joint openings and solution channels in rocks decreases rapidly as the depth increases.

Assuming, for simplicity, and as fairly representing experience, that the water-yielding capacity of a given kind of rock varies inversely as the depth, and that q is the maximum yield per square foot of surface at a depth 0 to 1 foot below the water table. The

quantity q corresponds to Slichter's "transmission constant" and its value can be determined from the measured yield of shallow wells in the given location and kind of rock. Then the yield per square foot of surface at a depth h below the surface of the water table may be expressed by the equation

$$q_h = q/h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The total yield from one side only of a vertical surface one foot wide intersecting the water bearing rock from a depth h_1 to a depth h_2 will be to

$$Q' = q_1 \int_{h_1}^{h_2} \frac{dh}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

If p is the ratio of the perimeter of a tunnel to its height, or the ratio of the perimeter of a well to its diameter, then letting Q_1 equal PQ^1, where Q_1 is the total yield of a horizontal tunnel per foot of length, or the total yield of a well between any chosen depths per foot diameter, there follows by intergrating

$$Q_1 = q_1 p \, (\log_e h_2 - \log_e h_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

If it is desired to determine the total yield of a well from surface to bottom, h_1 should be taken as unity to correspond to the conditions assumed in fixing the value of q_1.

As a practical example, statistics gathered by E. E. Ellis, M. L. Fuller and others show the average yield for numerous shallow wells, on steep rocky areas, in granitic gneiss and schist of southeastern New York and Connecticut, to be about 4 gallons per minute for 6 inch wells with an average depth in rock below the water table of about 150 feet. The perimeter of a 6-inch well is 1.54 feet and the corresponding rate of yield for a flat surface one foot wide and 150 feet deep would be 2.60 gallons per minute. Solving (3) for q_1 under these conditions:

$$q_1 = \frac{2.6}{\log_e 150} = 0.52 \text{ gallon per minute}$$

It should be noted that only wells with little cover of sand or other unconsolidated material have been considered in selecting the value of Q_1 to determine q_1. Shallow wells in rock but with

deep cover of loose material often receive much of their supply from above the rock, and may indicate water quantities in excess of the transmission capacity of the rock itself, even at the surface where it is most fissured.

Let it be required to determine the amount of water that will probably be encountered in a tunnel 15 feet high and with 60 feet perimeter, at an average bottom depth of 250 feet below water table in granitic gneiss and schist, the length of the tunnel in rock being 11,000 feet. Using the value of q_1 above determined, and solving (3)

$$Q_1 = 0.52 \times 4 (\log_e 250 - \log_e 235) =$$
0.135 gallon per minute per foot of tunnel

Let Q equal the total infiltration to the tunnel, the length being 1. Then $Q = Q_1 L = 1485$ gallons per minute. This indicates that provision should be made to dispose of 1485 gallons per minute, or something over 2,000,000 gallons per day.

Experience in constructing the tunnel for which data are above given showed an average yield for some time of 1,000,000 gallons per day, and larger yields at times, approaching the amount above calculated.

Like most hydrologic methods and formulas, this one is not infallible, and its application requires intelligent discrimination in selecting surface-well data to determine q_1. It is intended to apply to fissured rocks such as granites, trap, basalt, gneisses, schists, and sometimes limestones and dolomites. It is possible that a large water vein may be encountered which will greatly increase the yield, but the probability of this decreases with increased depth.

In the case of unconsolidated materials and sandstones, the water transmission capacity may decrease little or none with increased depth, and formula (3) does not apply. The quantity of water obtainable from such deposits by means of wells, tunnels and infiltration galleries can be calculated by existing formulas given in various books on water supply.

In the case of shales, experience shows that the water-bearing fissures decrease very rapidly with increased depth, in fact little or no water is generally obtained in dense shales at depths more than 50 to 100 feet below the rock floor. Observations by the author in the case of many deep wells in Hudson River shale as to the relation of depth to yield is fairly represented by the formula

$$Q_1 = 2q_1 p \left(\frac{1}{h_1} - \frac{1}{h_2}\right) \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

This is derived on the assumption that both the number and size of connected fissures in shale decrease as the depth increases, or that the transmission capacity per unit of surface varies inversely as the square of the depth. For example, if a 6-inch well extending 20 feet below water table in shale yields 6 gallons per minute, then solving for $q_1 p$ in (4) by taking $h_1 = 1$ and $h_2 = 20$

$$q_1 p = \frac{6}{2\left(1 - \frac{1}{20}\right)} = 3.15$$

To find how much the yield of this well will be increased by extending it to a depth of 100 feet, we have from (4)

$$Q_1 = 6.30\left(1 - \frac{1}{100}\right) = 6.24$$

The yield of the well would be increased $\frac{1}{4}$ gallon per minute by extending it from 20 to 100 feet depth.

The formula indicates that half the water obtainable will be found in the first foot of depth in shale rocks.

This result is certainly of the right order, since experience shows that nearly all the water obtainable in Hudson River shale is often found within 5 feet below rock surface. A number of wells to depths of over 1000 feet in shale in the vicinity of Albany, N. Y., afford their entire yield very near the surface and refute the popular belief that water can always be found in abundance if only a well is drilled deep enough.

Formula (3) can be applied in a manner similar to (4) to determining the probable increase of yield with depth for wells in granite, etc., and indicates little expectancy of increased yield at depths exceeding a few hundred feet, which also is verified by experience.

Formulas (3) and (4) are intended more especially for application to tunnels and mines rather than to wells, since the surface area of ground water interception in the case of the former is much larger and will generally approximate average conditions more closely than for isolated wells.

IMPROVING BOILER EFFICIENCY AT THE DAYTON WATER WORKS

By H. C. Wight

In the pumping station of the Dayton water works there are two 415-horse-power boilers having an allowable steam pressure of 155 pounds and two 465-horse-power boilers with an allowable pressure of 167 pounds. All were equipped with automatic steam dampers some time ago, which were expected to increase the efficiency of operation. In fact, they are generally satisfactory with hand-fired boilers.

About two years ago the two larger boilers were equipped with automatic stokers, superheaters and soot blowers. A suspicion arose that the efficiency of these boilers could be increased. Bailey flow meters have accordingly been installed on the boilers having the stokers, and the suspicion has been shown by the charts taken by the meters to be well-founded. These charts indicate that the automatic dampers were responsible for burning about 5 per cent more coal than was necessary.

Two of these charts are reproduced diagrammatically to show what is indicated by them under the conditions existing in the Dayton plant. The record kept near the center of the chart gives the stack temperature, in degrees Fahrenheit. This is the source of the largest waste in boiler practice. The record shown on the outer part of the chart by a dotted line is in red in the original chart and gives the rate of steam flow compared with the boiler rating. It will be observed that the rate exceeds the boiler rating during a large part of the 24 hours. The solid lines record the draft and are in blue on the original chart. When the conditions are best the red and blue lines should coincide. When the blue (solid) line is above the red (dotted) line there is too much air. When the blue (solid) line is below the red (dotted) line there is too little air.

The chart of March 7, when the automatic dampers were in use most of the time, shows that at 8 o'clock in the evening the load was cut down. Nevertheless the stack temperature between 8

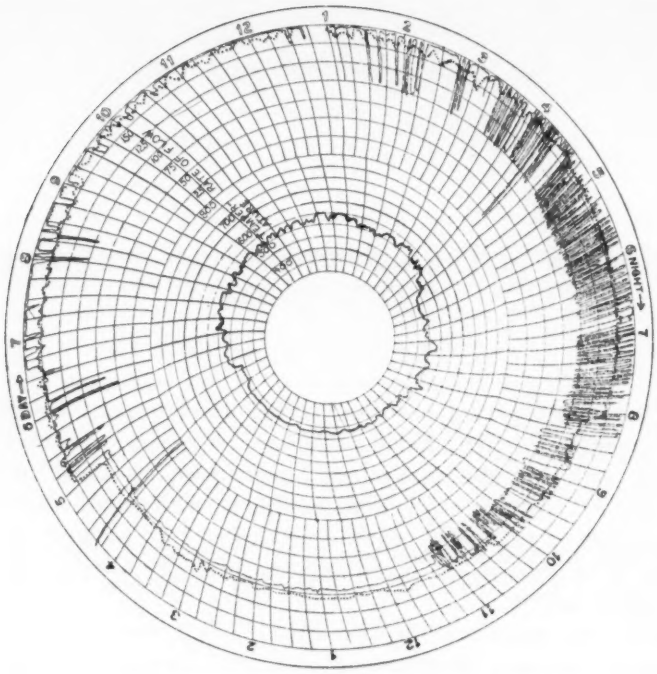


FIG. 1. BOILER METER CHART, MARCH 7, 1919

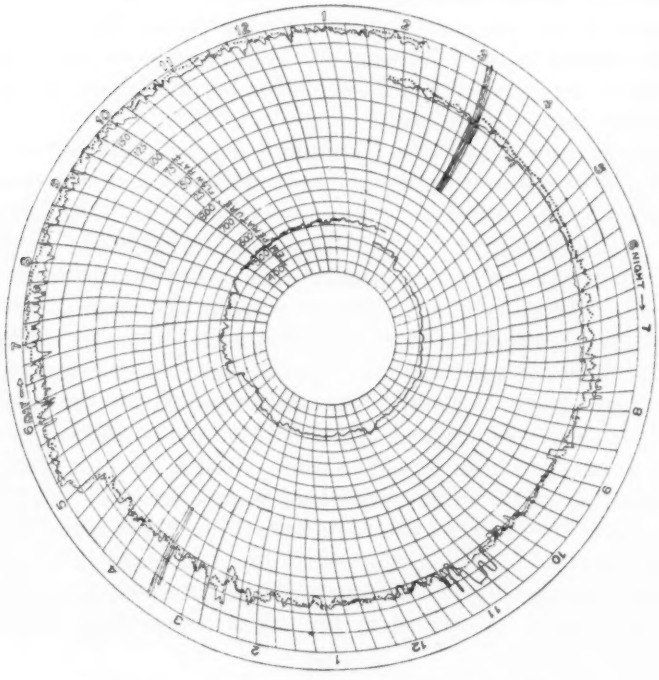


FIG. 2. BOILER METER CHART, MARCH 9, 1919

and 10 o'clock averaged higher than the average between 10 and 5 o'clock, although the gradual accumulation of soot on the flues would have made the conditions of temperature different had other features of the operation of the boilers not counter-balanced the effect of the soot. It should be observed that from 1 o'clock in the afternoon to 11 o'clock in the night of the period covered by this chart, the automatic dampers were in use, and during the remainder of the twenty-four hours, the boilers were run without them. At 4.20 in the morning the soot blowers were operated, as indicated by the long line in the draft record, and the stack temperature at once dropped 10° to 15° and remained down until the load was increased at 5 o'clock.

The chart of March 9, when the automatic draft apparatus was not used at all, shows the firemen maintaining a draft suitable to the rate of steam flow, with a correspondingly low stack temperature.

NOTES ON UTILITY AND PERFORMANCE OF COMPOUND METERS[1]

By Fred B. Nelson

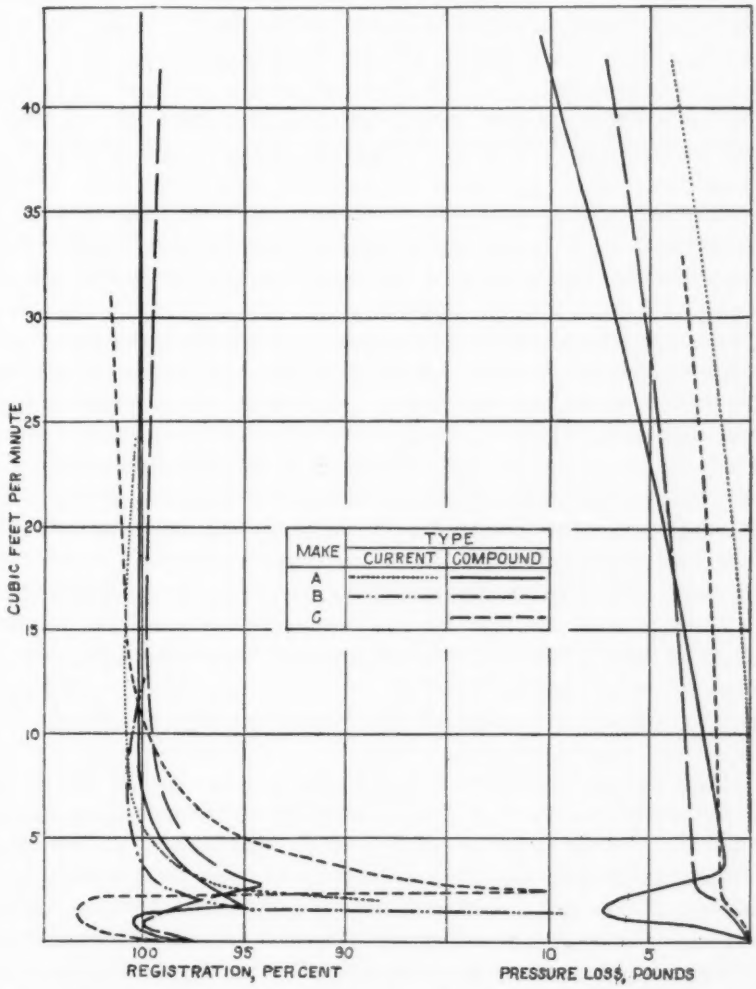
What the author has to say about compound meters is not based on experience with them in service nor on elaborate study of their principles and performance, and departmental tests and investigations in New York have not to date been sufficient to warrant positive conclusions or elaborate treatment of the subject. If, therefore, the author merely mentions some phases of the topic which will bring out discussion from those who have had experience in their design or use, the purpose of the paper will be met.

The compound meter designed to obviate the loss of registration on flows too small for accurate measurement by the full-line-size meter consists essentially of three units, a large meter, a small by-pass meter and a regulating valve so arranged that as the flow through the by-pass reaches a reasonable capacity of the small meter, the pressure loss actuates the regulating valve which opens and deflects all or part of the flow to the larger meter.

At the present time, compound meters as such are not on the approved list for use in New York City, but the advisability of their use in saving revenue on numerous large and often over-size installations has been considered, particularly in the past one or two years. Nearly a year ago, tests were made on fifteen current meters in service, from 3 to 6 inches, inclusive, in size, to determine the amount of under-registration in actual service. These tests were arranged by inserting a small meter in a by-pass around the outlet valve of the large meter; thus, with the outlet valve closed, the meters would be working in series, each under the variation of rates produced by the actual service demand. The duration of the tests was from one week to five months of continuous operation. The results showed a total monthly registration on test meters of 828,229 cubic feet against 651,642 by the service meters, an under-registration of 176,587 or 21.3 per cent, representing on the 15 meters

[1] Read before the New York Section on February 19, 1919.

a revenue loss of $176.58 per month or $2119 per annum. Out of the 15 service meters, three over-registered 2, 4 and 11 per cent respectively. The under-registrations of the other 12 ran from 6 to 95 per cent. While the results secured on these meters may not be accurately representative, they are at least suggestive of the



Tests by Department of Water Supply, Gas and Electricity, New York City, of performance of four-inch compound meters, including comparison with performance of current meters of the same makes and size.

possible loss of revenue being sustained by lack of proper compounding of the 3500 odd meters 3 inches and larger now in service in the city. If the tests were representative of the actual conditions, the annual loss to the city would be upwards of one-half million dollars.

Other and more striking examples might be mentioned as illustrations of the possible saving to be secured by compounding. In one instance, the replacing of an old 8-inch standard meter by a compound combination of an 8-inch current meter, 8-inch regulating valve and 2-inch by-pass meter, resulted in raising the monthly registration from 1645 to 56,480 cubic feet, a saving of $650 per year.

Tests for accuracy and pressure loss at varying rates of flow have to date been made by the department on five makes of 4-inch compound meters. By a study of the resulting curves some features of possible improvement are suggested.

On all meters tested, the by-pass meter, usually about $\frac{3}{4}$ inch, is sufficiently small to register with accuracy, very minute flows, as low as 0.01 cubic foot per minute, but, on the other hand, it does not seem to be of sufficient capacity to carry the load to a rate which can be accurately registered on the large meter before reaching the pressure loss at which the regulating valve opens, resulting in an under-registration immediately following the opening of the valve and between the total rates of about $1\frac{1}{2}$ and 3 or 4 cubic feet a minute. It would seem from the average curves obtained, that accuracy at these rates, which could easily represent a large portion of actual use or continuous fixture leakage, was being sacrificed in order to secure registration of the extremely small flows of say less than 0.1 cubic foot per minute which on all services of this size are relatively unimportant.

Under-registration of such service flows may also be due in a large measure to the very gradual opening of the regulating valve, while the by-pass meter, subjected, as in some makes, to the same pressure difference, continues to operate on high flow, leaving the increase of flow only to be deflected through the larger meter with but slight, if any registration. For example, assume that the rate is increased to 2 cubic feet a minute with the regulating valve at the point of opening and the by-pass meter registering the total flow accurately. Then if the rate is increased to $2\frac{1}{2}$ cubic feet per minute with a slight opening of the regulating valve and no decrease

in pressure loss the by-pass meter will continue to register a full 2 cubic feet accurately and the $\frac{1}{2}$ cubic foot only will pass through the current meter without registration so that the combination would under-register $\frac{1}{5}$ or 20 per cent. Instead of this gradually opening regulating valve it would seem that one designed to more abruptly deflect practically the full flow from the by-pass to the main line would be preferable, this action to be accomplished either by mechanical closing of the by-pass or a sufficient valve opening to materially decrease the pressure loss and the resulting by-pass flow.

On some compound meters the stem of the regulating valve carries within a cylinder a small, close fitting piston, one side of which receives the water pressure of the main tending to hold the valve closed while the other side of the piston is open to atmosphere. If this piston were so designed that a slight opening movement of the valve would allow water pressure to replace the atmospheric, an opening impulse would be given the valve tending to suddenly reduce the pressure loss, thereby transferring a large portion of the by-pass flow to the large meter and favoring its more accurate registration. Possibly meter manufacturers can point out the impractical phases of this proposition.

A valve of this type partially open to atmosphere has a tendency to open more easily under low main pressure, thus deflecting the flow to the large meter at a lower rate than when working under a high main pressure, a matter which is probably compensated for by varying the weight of the valve.

From these more or less superficial observations, the following improvements are suggested:

1. An increase in the capacity of the by-pass meter to cover all service rates too small for accurate registration on the main line meter and at a pressure loss that is sufficiently low to avoid opening of the regulating valve until the flow is well within the range for accuracy on the large meter.

2. A regulating valve so designed as to open more abruptly with either simultaneous closing of the by-pass or with a sufficient reduction of pressure loss to materially relieve the by-pass meter and deflect the bulk of its flow through the main line, thereby setting up a flow through the main line sufficient for accurate registration on the large meter.

TREATMENT OF WATER TO PREVENT THE GROWTH OF CRENOTHRIX[1]

By F. C. Amsbary

The Champaign-Urbana Water Company takes its supply from tube wells 160 feet deep. The water in its raw state has an iron content of 2 parts per million. Six years ago the company installed an iron removal plant of 2,000,000 gallons daily capacity. The filter plant consists of four units. The construction is of reinforced concrete.

The company has been bothered with crenothrix and has used mechanical and chemical means of prevention with but little success. Last summer, at the suggestion of W. F. Monfort, liquid chlorine was tried with great success. The crenothrix growth had possession of the filter beds, rendering them inefficient; the reservoirs were foul with it, requiring frequent cleaning. The distribution system was supporting its proportion. Since pipes were stopping up all around the city, the situation was serious indeed.

The introduction of chlorine into the raw water as it enters the filters had an immediate effect on the crenothrix, causing it to disappear, and since the use of chlorine began the water has been free from the pest. The filter beds are easily kept sweet and clean, the efficiency is 95 per cent, and no growth can be detected in the basins or distribution system. It has been a great relief, for complaints which were numerous before its use have ceased entirely since chlorine has been employed. About 4½ pounds per million gallons are used. The chlorine is introduced into the water by means of a Wallace and Tiernan manual-feed apparatus.

DISCUSSION

F. C. Amsbary: The company was bothered with crenothrix before the filter plant was installed. For a time after the filters were put in operation it almost entirely disappeared. The presence of iron

[1] Read before the Illinois Section, March 25, 1919.

in water is very favorable to the growth of crenothrix, therefore when the iron was removed it died and very little, if any, was observed. The crenothrix, however, did continue to thrive in the raw water and gradually got a footing in the sand beds of the filters, rendering them less efficient in the removal of iron. This is possibly the reason for the growth getting a start in the distribution system.

C. B. BURDICK: In Winona, Minn., they had a great deal of trouble from iron and crenothrix. They recently turned water from new artesian wells into the supply from shallow wells formerly used and mixed it half and half. They have now been operating about three years and the trouble seems to be almost entirely eliminated, although the artesian water contains the same amount of iron as that from the shallow wells.

A. N. TALBOT: The author's fear that this cure may not be permanent indicates that he may have been looking into the history of the water supply. When the plant was built a good many years ago, the water was pumped from the wells into a basin which was covered with a roof, and after a time considerable trouble developed. Professor Forbes thought it was worth while to remove the roof of the reservoir, since the crenothrix thrives in the dark. The roof was taken off and it was reported that there was considerable improvement thereafter. Later the trouble began to reach other parts of the system and the growth in the mains became troublesome. Then the author took up the matter of an iron removal plant. That worked well for a time, until here again the crenothrix changed its habits and began to grow down in the sand of the filter and there was a recurrence of the trouble. Still later a roof was put on the clear-water basin to overcome trouble with worms. Mr. Monfort may think that after a while the crenothrix will get accustomed to the presence of chlorine and will thrive on it, and the author will have his old trouble again.

CHLORAMINE AND CRENOTHRIX[1]

By W. F. Monfort and O. A. Barnes

The application by Rideal of chloramine as a sterilizing agent for water and sewage has led to renewed interest in the reactions of chlorine with ammonium salts and with ammonia. Much earlier work has since been repeated in the hope of developing control of the reactions and prevention of losses. The fundamental work of Raschig has been commonly overlooked.

Chloramine (chloroamine, monochloramine) is formed[2] by treating dilute solutions of hypochlorites with dilute ammonia, which Raschig expresses in the following equation, $NH_3 + NaOCl = NH_2Cl + NaOH$.

By subjecting such a mixture to distillation in a vacuum at about 40°, after adding $ZnCl_2$ solution to remove free ammonia and sodium hydroxide, there is obtained a concentrated solution whose analysis yields values agreeing with the formula NH_2Cl. From concentrated mixtures under greatly reduced pressure, chloramine is often obtained as pale yellow globules floating in the aqueous distillate. Because of instability in concentrated form, no attempt was made to obtain it pure. Chloramine escapes readily from its aqueous solution, has the odor of nitrogen trichloride and vigorously attacks the eyes. (C. A. 2, 1533.)

With ammonia and sodium hypochlorite in equivalent amounts (in $\frac{1}{3}$ normal solution) there is some decomposition; liberation of nitrogen or reversion to ammonia is hastened by the presence of hydroxyl ions.

If calcium hypochlorite in equivalent amounts be substituted for sodium hypochlorite the reaction may be thus understood:

$$2CaOCl_2 + 2NH_4OH = 2NH_2Cl + CaCl_2 + Ca(OH)_2$$

With strong solutions reacting there is here a tendency to produce nitrogen trichloride and nitrogen.

It is apparent that a solution of chloramine of only relatively low concentration can be prepared; that if stronger solutions of bleach

[1] Read before the Illinois Section, March 25, 1919.

[2] Raschid, 1907, Chem. Zeit., 31, 926.

and of ammonia are used, the reaction must take place in the presence of considerable amounts of diluting water.

Experiments were undertaken to determine the stability of chloramine solutions of different strengths. There are two methods of determining the velocity of the reaction: (1) the content of chloramine remaining in the solution after the lapse of each period of test, and (2) the total volume of nitrogen evolved in a gas burette at the end of each period. The results of the experiments are given in table 1.

TABLE 1

Decrease in available chlorine of a chloroamine solution

HOURS	CHLORINE	NITROGEN EVOLVED FROM 50 CC. SOLUTION	HOURS	CHLORINE	NITROGEN EVOLVED FROM 50 CC. SOLUTION
	p.p.m.	*cc.*		*p.p.m.*	*cc.*
0	1003.443	0	23	860.09	
1	1003.443	0	24	850.00	0.51
2	1003.443	0	25	850.84	
3	1003.443	0	26		0.63
4	984.68	0.01	28		0.69
5	980.86	0.05	29	841.28	
6		0.09	32		0.73
7	975.12	0.12	51	707.44	
8		0.15	54		1.6
8½	965.56		73	554.48	
10		0.17	76		2.5
11½		0.27	98	430.20	
17	889.08		101		3.25
19	879.52		145	296.36	
20		0.37	148		4.10
21	869.96		170	213.00	
22		0.44	173		4.50

The amount of nitrogen evolved is a function of the rate of disappearance of available chlorine, which finds expression in the equation:

$$3NH_2Cl = N_2 + NH_4Cl + 2HCl$$

A series of determinations was made of the rate of decomposition of chloramine solutions of different strengths, as given in table 2.

The most concentrated solution used (10.2 grains available chlorine per liter, or 0.2877 normal) foamed vigorously at the start; the succeeding one less, and so through the several series, with the velocity

of the decomposition diminishing in each case as the period of experiment was prolonged, as indicated both by the volume of gas evolved and the concentration of available chlorine. A precipitation of cal-

TABLE 2

Decomposition of chloroamine solutions

HOURS	SERIES	AVAILABLE CHLORINE		HOURS	SERIES	AVAILABLE CHLORINE	
		Parts per million	Loss			Parts per million	Loss
0	1	10200	0	0	2	8160	0
1	1	7275	28.68	1	2	7388	9.46
2	1	6900	32.35	2	2	7013	14.06
3	1	6413	37.13	3	2	6619	18.88
7	1	4875	52.26	7	2	5438	33.36
11	1	3844	82.90	11	2	2194	45.76
24	1	1744	82.90	24	2	2194	73.11
32½	1	1125	88.97	32½	2	1819	77.71
53½	1	300	97.06	53½	2	469	93.03
0	3	6120	0	0	4	4080	0
1	3	5513	9.92	1	4	3750	8.09
2	3	5250	14.21	2	4	3638	10.83
3	3	4988	18.50	3	4	3516	13.82
7	3	4350	28.92	7	4	3169	22.33
11	3	3788	38.10	11	4	2897	29.00
24	3	2175	64.46	24	4	2016	50.59
32½	3	1538	74.87	32½	4	1603	60.71
53½	3	563	90.80	53½	4	863	78.85
0	5	3060	0	0	6	2040	0
1	5	2775	9.31	1	6	1875	8.09
2	5	2700	11.76	2	6	1844	9.61
3	5	2625	14.22	3	6	1813	11.13
7	5	2391	21.86	7	6	1706	16.37
11	5	2231	27.09	11	6	1638	19.71
24	5	1688	44.84	24	6	1350	33.82
32½	5	1397	54.35	32½	6	1194	41.47
53½	5	853	72.12	53½	6	863	57.69

cium hydroxide varied in the series of reactions from very heavy in the first to relatively slight in the last.

From these data it is apparent why the application of chloramine should be made under such conditions that the concentration of the mixed reagents shall not exceed 1 p.p.m. A solution even of this

low concentration should not be stored longer than twenty-four hours and is best made up as used. Not only do hydroxyl ions in solutions prepared from bleach and ammonia reduce the stability of chloramine; their reactions with bicarbonate ions in the treated water convert the latter to bivalent carbonate ions, and cause a copious precipitation of calcium carbonate.

$$\overline{OH} + \overline{HCO_3} = H_2O + \overline{\overline{CO_3}}$$

This has resulted in incrustation of feed pipes and of lines carrying the treated water from the point of applying chloramine.

Some experiments were carried on in preparing chloramine with dilute chlorine water and dilute ammonium compounds, NH_4OH, $(NH_4)_2$, CO_3, etc., with varying results. While it is theoretically possible that some chloramine might be formed under these conditions, the diverse possibilities of reaction, indicated by Bray and Dowell[3] offer little hope that direct addition of ammonia and liquid chlorine to a water can be so controlled and the distribution of the sterilizing agent be made so efficient as to insure the full chlorine equivalent in chloramine.

Rideal[4] found that when chlorine was introduced into sewage it was rapidly consumed, but that after free chlorine had entirely disappeared there persisted a strong germicidal power. The same results were obtained when bleaching powder was added to a water containing a small amount of ammoina. It seemed probable that the ammonia did not increase the oxidizing power of chlorine, since readily oxidizable organic matter in water absorbed much less chlorine from ammonia and hypochlorite than from hypochlorite alone. Furthermore the bleaching effect on dyestuffs indicated that ammonia and bleach together had only two per cent of the oxidizing (bleaching) power of hypochlorite alone. While chloroamine has little oxidizing value it is still able to displace iodine from potassium iodide, giving the usual starch-iodide reaction; its chlorine can be precipitated by silver nitrate, and its ammonia equivalent determined with the strongly alkaline Nessler's reagent. Rideal showed that chloroamine has a phenol coefficient of 6.6, three times that of chlorine.

According to Dakin[5] the germicidal value of hypochlorite in

[3] Bray and Dowell, J. Am. Chem. Soc., 39, 905.
[4] Rideal, J., Roy. San. Inst., 31, 33, 1910.
[5] Dakin, Proc. Roy. Soc. Lond., Series B. 89, B. 614, pp. 232-251.

sewage is due to chloroamine derivatives produced by the action of chlorine on amino acids and proteins. The proteins present in sewage contain amino groups which may react with chlorine to form substituted chloramines containing the NCl group. When chlorine or bleaching powder is added to sewage, there may occur a primary oxidation and a secondary formation of toxic chloramines from the reaction of aminobodies. When chloroamine is added to sewage the initial rapid oxidation is eliminated, and the germicidal action begins at once. Dakin attributes the latter action to the chloramine group.

It is possible that proteins of living cells may so react, and that the killing of microorganisms by hypochlorites is due to chemical changes of this character within the living cell, either by direct action of the germicide or by secondary products of similar nature.

Action of chloroamine on crenothrix. The pronounced success of chloramine treatment in the destruction of vegetative bacterial cells suggested its application to an especially troublesome water pest. Crenothrix is one of the iron bacteria, so called, because of their occurrence in iron-bearing waters. It belongs with the true bacteria (eubacteria). Its cylindrical cells are united in unbranched threads, enlarged toward the free end, covered with a thick sheath which becomes infiltrated with ferric hydroxide. Reproduction takes place by the division of the cells in three planes, with the formation of round gonidia. It is still in dispute whether the organism receives its energy from oxidation of ferrous to ferric iron[6] or whether the separation of ferric hydroxide is an independent mechanical phenomenon not connected with the life processes of the cell.[7]

It occurs in numerous sections of the State of Illinois in surface waters and in some well supplies, as at Freeport and Champaign-Urbana. In 1917 the latter supply was rendered unsightly and unfit for some domestic purposes by the development and decay of the organisms in the distribution system.

In carrying on the experiments upon the action of chloroamine on crenothrix it was impossible to plate the treated water on media, as is done with ordinary water bacteria, since no medium had been discovered upon which it can be surely grown.

Chloroamine was first tried on water from the University well, which furnishes an iron-bearing water already inoculated with

[6] Winogradsky, Proc. Zool. Soc. Lond., 1913, Pt. 2, 430.

[7] Mollisch and Ellis.

crenothrix. The first tests demonstrated in duplicate 800 cc. samples the germicidal action of 1 p.p.m. available chlorine in freshly prepared chloramine. At the end of a week there was no growth in the treated samples, while control samples, untreated, showed an abundant reddish growth on the bottom.

In later series 11-liter samples were used: one set with chloroamine in amounts equivalent to 0.5, 0.75 and 1 p.p.m.; a second with bleaching powder equivalent to 1 p.p.m. available chlorine. Control samples stored without treatment showed an abundant growth at the end of one week. At the end of three weeks water treated with bleach alone showed a velvet growth, identified as crenothrix by microscopical examination. Samples treated with 1, 0.75, and 0.5 p.p.m. of available chlorine in chloramine gave no growth at this time nor within six months thereafter. The odor in the last mentioned samples was pleasant, noticeably better than that of untreated or bleach-treated samples; nor was there at any time a noticeable taste, save in those which had received the largest application of chloroamine (1 p.p.m.), in which there was a slight flavor as of chloroamine. In the controls and in the bleach-treated waters, the taste and odor were offensive.

These tests were repeated with similar results. A chloroamine addition corresponding to 0.5 p.p.m. was effective in preventing development of crenothrix; the residual matter did not become offensive even after prolonged storage.

These results, gotten in a small way, indicated that the acute troubles arising from crenothrix in iron-bearing waters may be eliminated by the germicidal action of chloroamine, thus reducing the problem of treatment to one of iron removal without complications. It was intended to apply the experiment in a large way at the 2,000-000-gallon plant of the Champaign-Urbana Water Company immediately after the conclusion of the first experiments. There was at the time (January, 1918) difficulty in commanding a supply of ammonia and of reliable bleaching powder. The method substituted, the use of liquid chlorine, is described on page 194.

WATER SOFTENING FOR MUNICIPALITIES[1]

By Milton F. Stein

Hardness and its causes. The presence of salts of calcium and magnesium in water give rise to peculiar physical and chemical properties collectively spoken of as hardness. It is more easily defined in terms of chemical tests than in its popular conception, which at best is rather indefinite and to a large extent relative. A traveler will readily notice the difference in this respect between the water of one town and another, whereas the residents, through habit, probably do not consider the degree of hardness of their water supply objectionable. However, if through a lapse of treatment, there is a small rise in hardness in the water of a town accustomed to softened water, vigorous protests are heard at once.

Of the more general properties constituting hardness the failure to lather promptly stands first. This results in an undue consumption of soap and patience and in the formation of an insoluble lime curd, disagreeable to the skin in personal use, and rendering clothing greyish, streaked and stiff in laundry operations. Then there is the scaling in boilers, kettles and hot water pipes. The author also believes that hard water has a taste, or at least is less agreeable to drink than soft water. The individual constituents have special characteristics which should be briefly noted.

a. Calcium and magnesium bicarbonates are very similar in properties. They are very soluble in water, increasingly so with rising temperature up to about 170° F., where decomposition commences, with the formation of carbonic acid and normal carbonates, the latter, relatively insoluble, being precipitated. Hence, these two salts are said to cause temporary hardness, being removed by heating or boiling. Their precipitation in boilers forms soft scale.

b. Calcium sulphate is very soluble in cold water but becomes decreasingly so on heating, particularly with increasing pressure. Hence, when concentrated by evaporation in steam boilers, it soon reaches the precipitation point, and forms a hard scale, so that it is

[1] Read before the Illinois Section on March 25, 1919.

bound to be troublesome even if present in small quantities, unless the boilers are frequently blown off, which is both troublesome and expensive.

c. Magnesium sulphate is not only very soluble in cold water, but becomes increasingly so on heating, and therefore does not itself form scale. It hinders the lime-softening process, however, combining with lime to form the objectionable calcium sulphate. Under high boiler pressures it will combine with any calcium carbonate present, resulting in a double precipitate of magnesium carbonate and calcium sulphate.

d. Calcium and magnesium chloride are the least important of the hardening constituents. Being extremely soluble, they do not cause trouble by incrustation.

The process of water softening. On adding slacked lime to a hard water, the following reactions take place:

(a) $CaCO_3$, $H_2CO_3 + Ca(OH)_2 = 2CaCO_3 + 2H_2O$
(b) $MgCO_3$, $H_2CO_3 + Ca(OH)_2 = MgCO_3 + CaCO_3 + 2H_2O$
(c) $MgCO_3 + Ca(OH)_2 = CaCO_3 + Mg(OH)_2$

In equation (a), the calcium carbonate formed is only soluble to the extent of about 13 p.p.m. and so precipitates out. The magnesium carbonate in equation (b), being soluble to the amount of about 100 p.p.m., remains in solution to that extent, and requires additional lime for its removal as the insoluble hydroxide. At least this is the theory, but in practice such ideal conditions do not obtain. The solubility of the calcium carbonate as precipitated from a solution seems to be greater than the amount which can be dissolved by shaking it in the powdered form with water. The presence of magnesium carbonate affects the solubility of calcium carbonate, as there appears to be a definite limit to the combined solubility of the two salts. The presence of other compounds, particularly sodium chloride, tends to increase these residual solubilities, and they are also affected by the temperature of the water. The magnesium hydroxide in equation (c), forms very slowly and is apt to assume a colloidal condition, and to return to its former state by absorbing carbonic acid from the air.

If either the sulphate or chloride of magnesium is present a further reaction takes place:—

(d) $MgSO_4 + Ca(OH)_2 = Mg(OH)_2 + CaSO_4$.

As will be seen, this consumes lime uselessly, since the calcium sulphate formed is the most troublesome kind of permanent hardness, and the magnesium hydroxide will probably remain in colloidal solution.

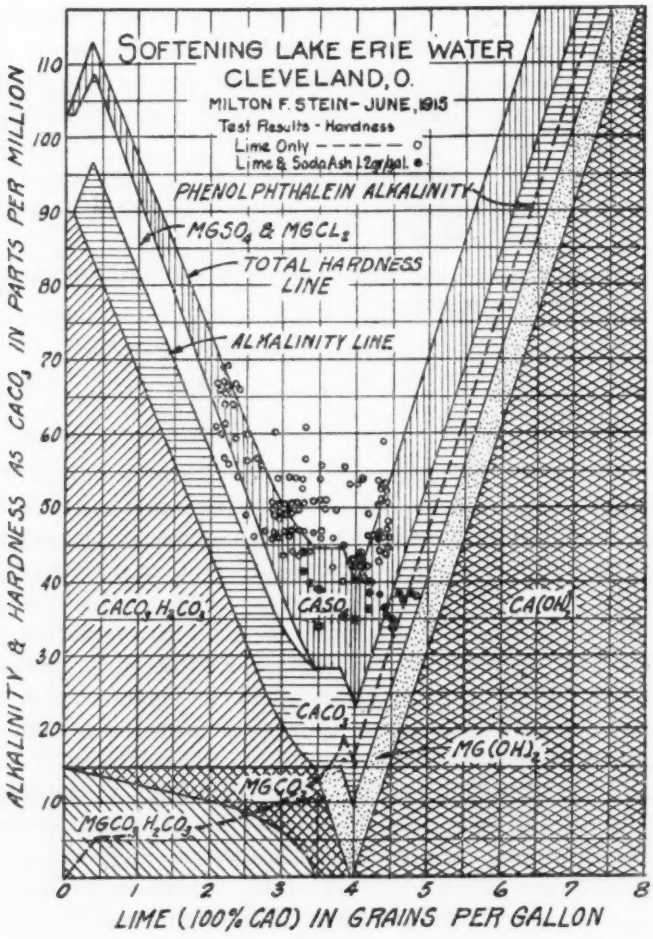

SOFTENING LAKE ERIE WATER
CLEVELAND, O.
MILTON F. STEIN - JUNE, 1915
Test Results - Hardness
Lime Only - - - - - - o
Lime & Soda Ash 1.2 gr/gal. •
PHENOLPHTHALEIN ALKALINITY
$MGSO_4$ & $MGCL_2$
TOTAL HARDNESS LINE
ALKALINITY LINE
$CACO_3$ H_2CO_3
$CASO_4$
$CA(OH)_2$
$CACO_3$
$MG(OH)_2$
$MGCO_3$
$MGCO_3$ H_2CO_3
ALKALINITY & HARDNESS AS $CACO_3$ IN PARTS PER MILLION
LIME (100% CAO) IN GRAINS PER GALLON
0 1 2 3 4 5 6 7 8
10 20 30 40 50 60 70 80 90 100 110

FIG. 1

In figure 1 are shown graphically some of the results of an investigation on softening Lake Erie water at Cleveland, Ohio. This diagram shows the ultimate composition of the water after treatment with various amounts of lime, and on the assumption of a very

long reaction period. The vertical scale gives alkalinity and hardness as parts per million of calcium carbonate. The horizontal scale shows the amounts of lime added, in grains per gallon. It shows that the hardening constituents of this water consist of 15 p.p.m. magnesium bicarbonate, 75 p.p.m. calcium bicarbonate (these, combined, giving the water an alkalinity of 90 p.p.m.) and magnesium sulphate and chloride to the extent of 13 p.p.m., making the total hardness 103 p.p.m. Treatment with approximately half a grain of lime results in an increase of hardness, since the calcium carbonate formed is less than the saturation value and remains in solution. Larger quantities of lime result in progressively decreasing hardness. It will be observed that the lime reacts with the hardening constituents in proportion to their amounts, and that only that part of the lime which combines with the calcium bicarbonate does effective work, since in the case of magnesium bicarbonate, the soluble monocarbonate is formed, and the magnesium sulphate is replaced by the more objectionable calcium sulphate. By adding as much as 4 grains of lime some of the monocarbonate of magnesium can be precipitated as the hydroxide, in theory at least, while more than 4 grains causes an increase in hardness again. A judgment based on this diagram would place the optimum amount of lime at about 3 grains, which was borne out by the results of numerous large scale tests at the Cleveland experimental plant.

Lake Erie water lends itself particularly to such a diagrammatic representation because of its constant composition. For most waters, a series of diagrams or a model would be necessary, owing to variation of the constituents.

The rate of reaction in water-softening is quite slow. Some experimental results obtained with Cleveland water are shown in figure 2. The dots show the results of five tests run, using $2\frac{1}{2}$ grains per gallon of lime and with a water temperature of 20°C. The methyl orange and phenolphthalein alkalinities were determined by titration at short intervals. The arrangement of the points shows that a very definite law governs the course of the reaction. From the determinations and supplementary tests it is possible to surmise what is happening, as shown by the shaded areas. One point of interest not brought out by the main diagram becomes evident on plotting the logarithms of the values, as shown by the smaller diagram. There is a very evident break in the continuity, slope and curvature of the line at a point corresponding to log. alkalinity =

4.1, or alkalinity = 60. This suggests that the reaction is taking place in two steps, first, the reaction of the lime with the calcium salts; second, with the magnesium salts. The same thing may be noticed in connection with the soap test as applied to water con-

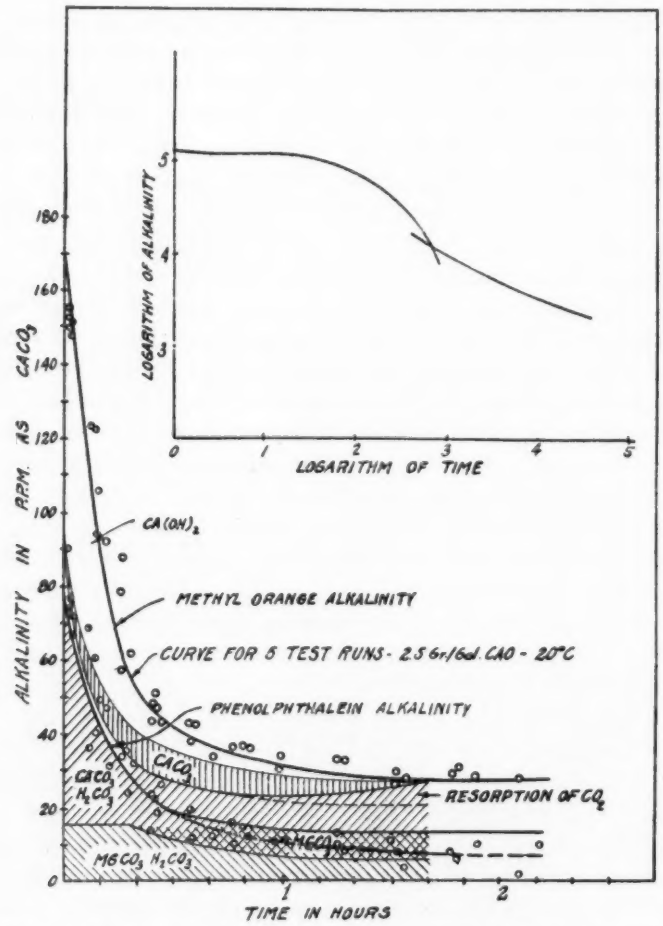


FIG. 2. RATE OF REDUCTION IN SOFTENING LAKE ERIE WATER

taining both calcium and magnesium compounds. There is a false end point when the calcium compounds have been displaced, followed later by the true end point when the reaction with the magnesium salts is completed. In all cases the magnesium compounds seem to be more sluggish in action.

Figure 3 shows the progressive softening reaction which took place in the Cleveland testing plant. The incoming water was measured by an orifice barrel, the lime solution being proportioned to it by a

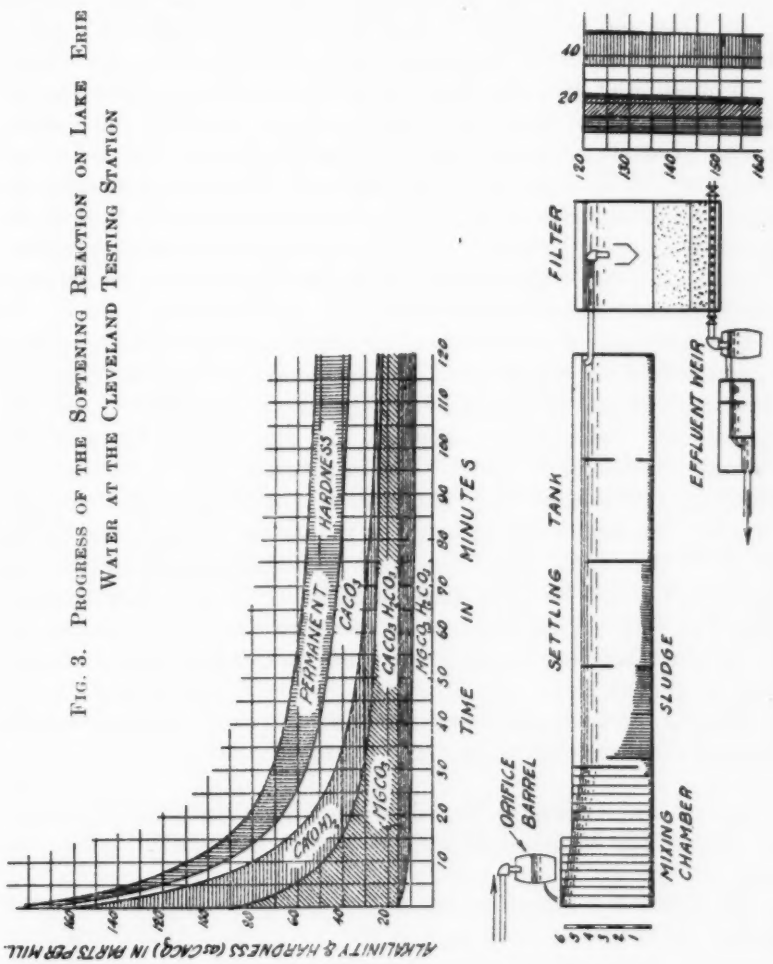


FIG. 3. PROGRESS OF THE SOFTENING REACTION ON LAKE ERIE WATER AT THE CLEVELAND TESTING STATION

float operated device. It then passed through an up-and-down mixing chamber into a settling tank. The depth of sludge deposit in the settling tank is indicated. From the settling tank it passed

through a mechanical filter of the usual type. There was usually a drop in alkalinity in passing through the filter, due to the precipitation of colloidal magnesium hydroxide and probably calcium carbonate by contact with the sand, as well as to a continuation of the softening process. There was some incrustation of the sand, as well as on plain and galvanized iron plates which were exposed continuously for 5 months in a stream of the softened and filtered water. This question of after-deposits is a serious one, and requires special study in individual cases. It causes trouble in the filter sand, underdrains, controllers, in meters, if galvanized inside, and in house plumbing. It is much more pronounced if the attempt is made to carry the softening to the limit than if a margin of safety is allowed, probably stopping just short of where the magnesium reaction begins. The addition of a coagulant during the latter portion of the settling period seems to act as a preventive.

The question of temperature in connection with the reaction period should be mentioned. The temperature factor in chemical reaction velocities is still a debated point. No formula of general applicability has yet been devised. In many cases the reaction velocity doubles for every 10°C. rise in temperature. Since the data heretofore considered were obtained at about optimum temperature for large scale water softening in the temperate zone, viz.: 20°C. or 68°F., this would mean that for a four-hour reaction period, an eight-hour period must be substituted at 10°C., or a sixteen-hour period at 0°C. Besides this there seems to be a sort of inertia to the reaction starting at all in extremely cold water, which is very discouraging at times.

Nothing has been said as yet regarding the use of soda ash for removing permanent hardness. The equations are:

$$\text{(a)}\quad CaSO_4 + Na_2CO_3 = CaCO_3 + Na_2SO_4$$
$$\text{(b)}\quad MgSO_4 + Na_2CO_3 = MgCO_3 + Na_2SO_4.$$

The reaction is sluggish when carried out in the cold. When the permanent hardness is low or is due to by-products of the lime process, the amount of soda-ash required is small and according to the law of mass action, the rate of reaction will be very slow. The magnesium carbonate formed in the case of magnesium salts is quite soluble and remains in solution. It will thus be seen that the use of soda ash in softening municipal water supplies is on a much more

precarious basis than in the case of boiler feed waters, and should not be entered upon without careful study and serious consideration.

The essential features of a softening plant. In the light of the foregoing and from other experience the requisites for carrying out a water-softening project successfully may be given.

1. There should be a very thorough analytical and experimental investigation which should cover particularly:

a. Analyses of the water in question and the variations to be expected therein.

b. The amount of reagent which can be successfully applied without causing trouble from after-deposits and incrustation. It is suggested that special attention be given to the point where magnesium reactions commence.

c. The question of whether it is advisable to attempt the removal of permanent hardening constituents in view of the additional burden which the use of soda ash involves.

d. The determination of the reaction period required under conditions of minimum concentrations of reagent and minimum temperature.

To interpret experimental data in terms of practical working conditions requires special experience based on contact with both phases of the problem.

2. There must be an understanding on the part of the municipal authorities that to accomplish its task a water-softening plant must be adequate throughout, and that there can be no skimping in the size of parts or in equipment or in operating expenses if satisfactory results are to be obtained. This holds more rigidly in the case of softening plants than in the case of ordinary water purification plants.

3. The principal design features of a water-softening plant should include:

a. Mixing chambers in which the softening reagents are thoroughly dispersed through the raw water, either by mechanical devices or by baffling. This mixing action also seems to accelerate the reactions.

b. Large settling basins whose capacity is based upon the reaction period as determined for conditions of minimum temperature. These basins will generally be several times as large as would be required or the regular type of purification plants.

c. Means for adding a coagulant solution either at the entrance to or near the exit from the settling basins, or at both points, to assist in starting and completing the softening reaction.

d. Mechanical filters of the usual type.

e. Special apparatus for properly slacking and feeding the lime solution into the water. Due to the tendency of lime emulsions to settle and to choke feed piping, this portion of the design requires particular care and study.

Special problems of the large softening plant. It is needless to give a description of the details of municipal water-softening plants of small or moderate size, since they conform so closely, except as to relative proportion of parts, to the usual type of mechanical filter plants. In the case of large installations, however, there have been certain special developments which may not be without interest.

As mentioned already, the settling basins will be quite large. The precipitate obtained will be voluminous, and special provision must be made for its frequent and expeditious removal. This requires that the settling capacity be divided into a number of units, and that at least one spare unit be provided so that cleaning will not interfere with the operation. Frequent clean-out valves must be placed in the floor of the basins. In the diagram of the experimental plant it was shown that at least three-fourths of the sludge collected in the first few feet of the settling tank, in fact it occasionally happened that this portion of the tank almost filled up in a week's time. It appears to the author that the first quarter-length of the settling basins should be made deeper than the remainder and equipped with automatic sludge removers of the continuous type, such as Dorr thickeners.

Since the sludge is composed essentially of the lime added to the water and an equal amount abstracted from the water, both in the form of calcium carbonate, it has been suggested that this sludge be reburned, thus recovering not only the lime added but an equal amount in addition. Mr. C. P. Hoover tried this experimentally at Columbus on a considerable scale, and small-scale experiments were made at Cleveland. The product is about 75 per cent water-soluble CaO, and slakes readily. The author made some preliminary studies for such a lime recovery plant, which included resettling tanks, special centrifugals and rotary kilns fired with producer gas. The estimated cost of reburned lime is about 75 per cent of that for commercial lime, based on available CaO. The reburning plant is quite a separate institution, requiring operators with a different type of training and experience. It is a manufacturing proposition, and this from present experience, does not lend itself readily to munici-

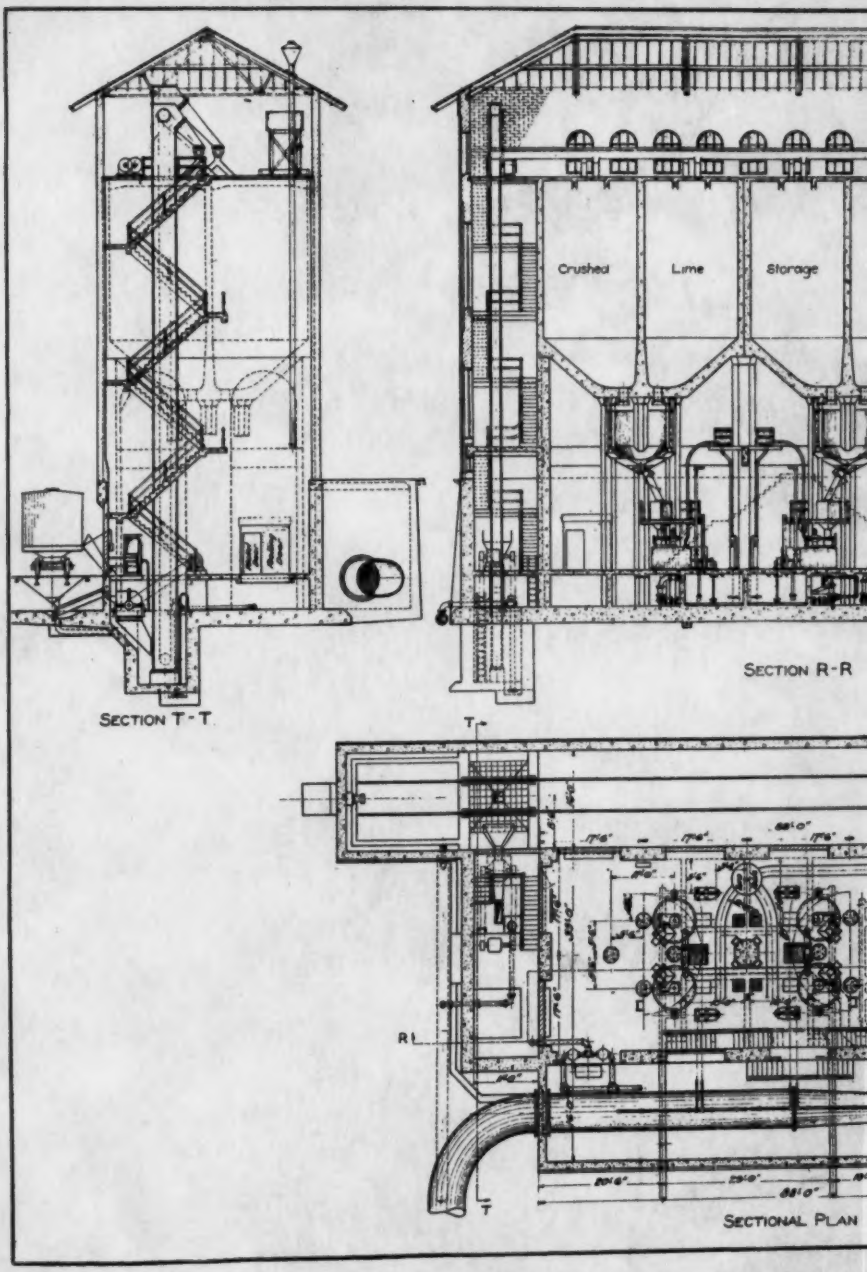
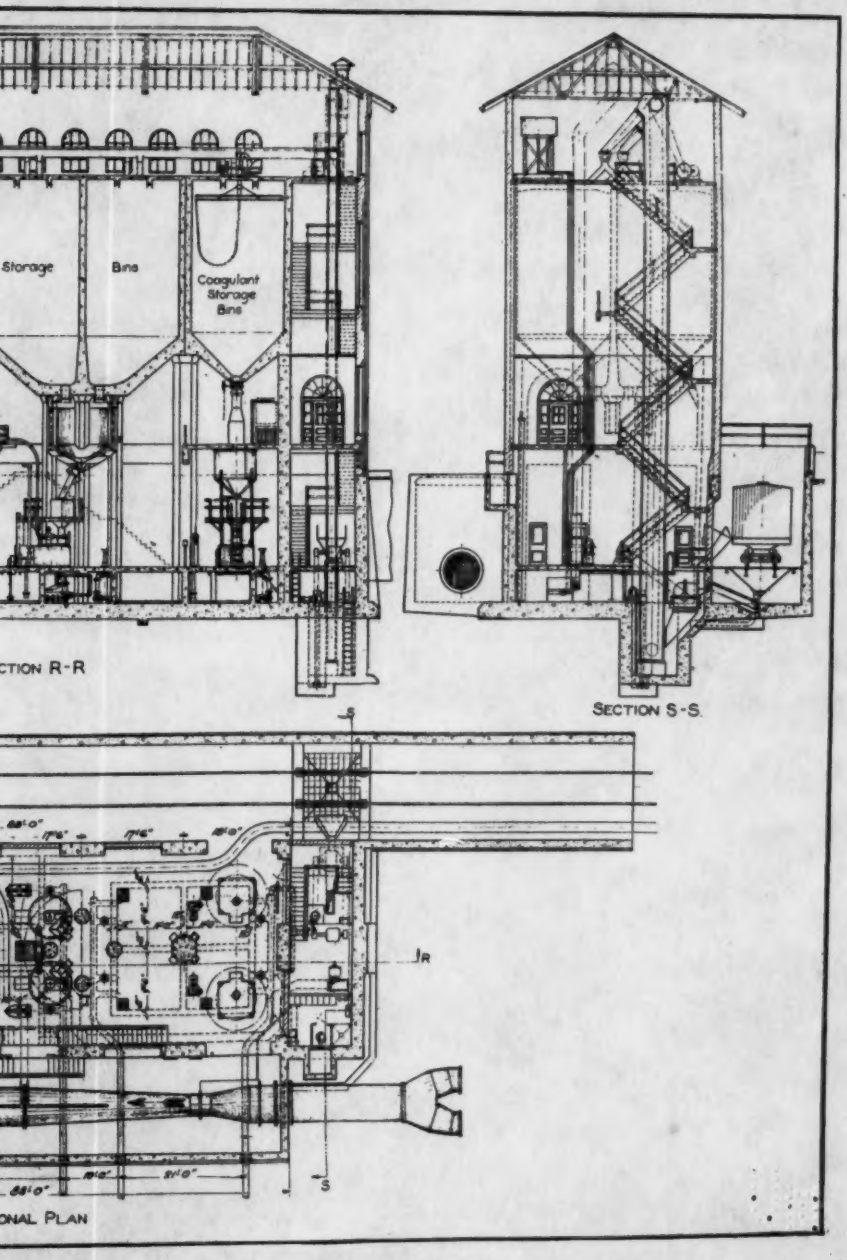


FIG. 4. CHEMICAL HOUSE, CLEVELA



, CLEVELAND FILTRATION PLANT

pal control. For this reason, and because the product is less easily handled, the advantages and disadvantages seem very evenly balanced.

Perhaps the most interesting phase of a large water-softening installation is the handling of the large quantity of chemicals used. Thus the Cleveland plant, operated at its capacity of 150,000,000 gallons daily, would require about 40 tons of lime per day. The arrangement of the chemical house is shown in figure 4. The lime arrives in box cars by way of a depressed siding on one side of the building. It is shoveled into a chute by power shovels, and falls into a crusher. The crushed lime is carried by elevator and screw conveyers to overhead storage bins, which have a capacity of 1500 tons of lime. Each four bins connect into one daily hopper and from this the lime flows by gravity to automatic scales.

The discharge of the scales can be automatically controlled to conform to the pumpage of raw water by means of the Venturi meter in the raw water line which passes through the chemical house. Figure 5 is a diagram of this device. From the raw water meter piezometer pipes lead to float tubes in the usual manner, the difference in head actuating differential gears, cams, etc., so that the resulting movement is directly proportional to the flow through the meter. A cord, with counterbalance to take up slack, transmits this movement to a traveling carriage *abcd*, which can move backward and forward on the rollers *r*, *r*, *r*, *r*, over an aluminum friction disc driven at constant speed by a synchronous motor. Mounted on the carriage is a small contact wheel. With no flow, the position of the carriage brings this wheel to the center of the disc, where it does not revolve. As the flow increases the carriage shifts and the contact wheel moves faster and faster. At each revolution this wheel closes an electrical contact, which, through suitable relays, actuates the automatic scales, causing them to discharge. These scales are really equal arm balances although entirely disguised by auxiliary levers, housing, and other complications. As soon as a scale is tripped, the automatic discharge gate opens and the lime flows out very quickly. This unbalances the scale beam, which tilts up, and in so doing closes the discharge and opens the feed gate. The feed gate, of the under-cut type, is gradually closed as the scale pan fills up and closes entirely when the predetermined amount has been reached. The scale is then ready to be tripped again by the contact wheel.

The scales discharge into slaking tanks, where the lime is slaked

with hot water, and agitated by motor-driven paddles. The slacked lime overflows into solution tanks where it is cooled and diluted with water, and is then pumped into the raw water.

The coagulant, either ferrous sulphate or alum, is handled in a similar manner, with minor modifications.

The Cleveland work was designed and built under the direction of R. Winthrop Pratt, consulting engineer, Frank H. Stephenson, designing engineer, Robert Sharp Jones, constructing engineer; the author being in immediate charge of the design of the portions

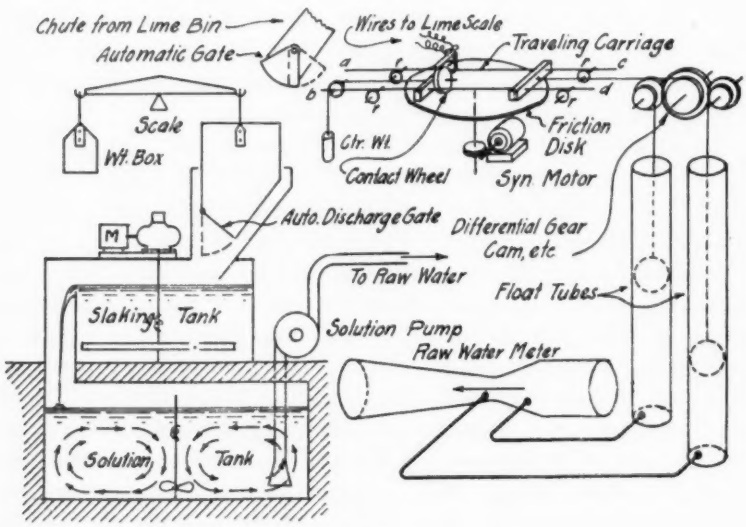


FIG. 5. LIME FEED CONTROL

herein described. The automatic scales were furnished by the Richardson Scale Company. The proportional feed device was built by the Builders' Iron Foundry, from sketches furnished by the author.

DISCUSSION

M. F. STEIN: In reply to questions the author stated that the rate of reaction should not be very different if a greater proportion of magnesium were present. The lime will react with both the calcium and magnesium bicarbonate in proportion to the amounts of these two present, but the magnesium carbonate as it is formed will

probably absorb some of the carbonic acid from the calcium bicarbonate, so that it would not be precipitated until the calcium bicarbonate had all been precipitated, but the rate of reaction would be essentially the same for any combination of the two salts, (this is more or less hypothetical).

A low temperature would slow up the reaction; the lower the temperature the slower the reaction.

If there were a superabundance of magnesium rather than calcium the magnesium would probably stay in solution in a colloidal form to quite an extent, more so at a low temperature than at a higher temperature. If there were more colloidal matter present, the calcium carbonate would tend to bring about a precipitation of the colloidal material by simple contact effect.

The experiments were run long enough to show that under carefully controlled conditions there is always a small amount of after-deposit. It was found that by restricting the dose to 3 grains of lime, or less, and not using soda ash it was practicable to cut down deposits to a negligible quantity.

There were bicarbonates present. In the lower part of the settling tank there would be carbonates present. After the water passed through the filters, and especially after it stood in the clear water tank a while, a certain amount of absorption would change some of the carbonates back into the bicarbonates.

There has been no trouble with bridging of the line in any part of the apparatus below the bin gates. There was sometimes bridging, or rather arching, just above the gates. In the scales the trouble was not very pronounced. With very fine, partly hydrated lime, the feed gates would not always shut off completely. Some of the lime would dribble in, hydrated lime flowing almost like water.

W. F. MONFORT: Scales of that kind sometimes overflow and deliver 2 or 3 bushels at a time. With hydrated lime there might be anywhere up to 200 pounds of fine material discharging from the scales at each operation.

M. F. STEIN: The scales discharged about every twenty minutes while the author was there. He left soon after the plant was put in operation. No softening had been attempted with the large plant, owing to lack of labor and other conditions. The author could not recall just how often the scales were figured to dump when the plant

is running, but believes it was about once every two minutes, for full capacity operation.

No attempt is made to warm the water by utilizing the heat from the lime slakers. Some of the details were taken from the St. Louis plant, because the same make of scale was used in both cases. The details for slaking the lime and heating the water are different.

THE LIME-SOFTENING OF WATER AND THE USE OF THE SLUDGES AS AN AID THERETO[1]

By W. A. Sperry

To soften water with lime is beautifully simple in theory but in practice there are difficulties that are both vexing and perplexing. Given ample control of the temperature and time element and a stable water of maximum softness results, otherwise the problems growing out of the tendency to form after-deposits may be of considerable moment.

Chemically, bicarbonates of calcium and magnesium are completely precipitated by the addition of lime water, the difficulties in the way of a complete realization of the lime water reaction being:

a. The fact that approximately 30 p.p.m. of the calcium carbonate produced are soluble, varying somewhat with the temperature and pressure conditions.

b. Whereas calcium is precipitated as a crystalloid, the crystal formation taking place slowly, magnesium is precipitated as a colloid. For complete removal it must be taken through the carbonate to the hydrate state, the reaction proceeding at a faster rate than does the calcium reaction.

c. Contrary to the general rule, calcium salts are more soluble in cold water than in warm, the completeness of reaction being greatly accelerated or seriously retarded by increase or decrease of temperature, as in changes from summer to winter conditions, with a corresponding change in the time element necessary.

These considerations form the fundamental elements on which the problem of design rests, the transgression of which may lead to serious difficulties.

For the municipal water softening plant this problem is more serious than it is for the smaller proprietary plant, in that the greater investment necessary in the municipal plant to insure the required "time of retention" element, as expressed in more land and larger plant, much increases the initial costs.

[1] Read before the Illinois Section, March 25, 1919.

The ideal condition for a lime softening plant is expressed in some form of the hot water softening system when designed with an ample time element. For the municipal plant this feature of using hot water is impossible of attainment and the design must be based upon the time required for the reactions to reasonably complete themselves under the winter condition. This involves for the mixing chambers, marked agitation and sufficient time for contact between the lime and the water under treatment before release to the settling basins, and for the settling basins, ample time for the completion of the reactions or the "curing" of the water before release to the filters or city mains.

The Grand Rapids mixing tank is a chamber built in two parts and arranged to be operated singly or in series. The smaller part provides for a retention period of twenty-two minutes at a 20,000,000 gallon rate and the larger part provides for a retention period of forty-five minutes at a 20,000,000 gallon rate. The smaller chamber is baffled so as to provide for four passes of the water in it, each pass 150 feet long, and the larger chamber provides for 49 passes, each 44 feet long—the baffles being in each case of the round-the-end or horizontal type. The larger chamber is so arranged that the water may be drawn off from it to the settling basins at either of the quarter-way intervals.

Table 1 gives results of experience at Grand Rapids with the changing seasons and illustrates the relations of time and temperature under discussion. With the river water running 220 p.p.m. total alkalinity and a magnesium content of 20 p.p.m. the temperature of the water ranging around 32° to 45°F., the alkalinity results from various parts of the mixing chamber given in table 1 are typical.

Here the significant thing is that under summer conditions with the temperature of the water ranging up around 70° to 80°F., the results attained after seventy minutes would be attained at the twenty-three to twenty-nine minute point with the alum dosage ranging 1 grain per gallon less and with greater rates of pumpage.

The essential feature of chemical application that seems to have crystalized into definite practice for lime softening at Grand Rapids is, that the lime should be added at the earliest possible moment to the water under treatment:

a. To insure, as far as possible, complete precipitation of calcium as carbonate.

b. To maintain sufficient caustic alkalinity or excess of lime so that there may be 5 to 10 p.p.m. remaining in the water at the point of discharge into the settling basins that the magnesium may be carried through the more soluble carbonate state to the less soluble, colloidal, hydrate state.

c. To delay the application of the coagulant as late as possible to the mixing chamber and still be assured of its thorough mixing with the water, before discharge to the settling basins, the alum playing a part not so much, chemically, to reduce alkalinity, as physically to scour out and precipitate colloidal matter.

As the result of the accordion-like use of the plant in the increase and decrease of the mixing chamber length, in the increase and decrease of the amount of alum used, and in the careful maintenance

TABLE 1

Alkalinity after different periods of detention in the mixing chamber
(parts per million)

	TOTAL ALKALINITY	PH. ALKALINITY	CAUSTIC ALKALINITY	NORMAL CARBONATE	MAGNESIUM
After 23 minutes	101	55	9	92	15.1
After 29 minutes	107	61	19	88	15.8
After 38 minutes	97	58	19	78	15.8
After 48 minutes*	91	58	25	66	15.1
After 54 minutes	59	31	3	56	11.8
After 70 minutes	59	32	5	54	10.9

*Alum applied here at the rate of 1.5 grains per gallon.

of residual caustic alkalinity, in an effort to cope with changing seasonal conditions to produce a water of uniform qualities and maximum stability, table 2 is presented, being a study by months of the results attained for the year 1917.

Note that the decrease in temperature with consequent retardation of the lime reactions is compensated for by increases in the time of contact and increase in the alum dose applied with maintenance of caustic alkalinity.

The application of this water to the filters and city mains has resulted, through a somewhat insufficient period for curing or maturing in the settling basins, in a very moderate amount of after-reaction trouble which has:

a. Much incrusted the filter sands, though the extent to which this has occurred lessens, by that much, trouble in the distribution system.

b. Limed up about 10 to 11 per cent of the 22,000 meters in service per year, Grand Rapids being some 98 per cent metered.

c Coated the service mains to a thickness of $1\frac{1}{16}$ to $3\frac{3}{32}$ inch in the last six years and caused some trouble, thereby, in the opening of valves and gates.

TABLE 2

Results, in parts per million, of lime softening during 1917

MONTH	RIVER WATER		NORMAL CARBONATE, SETTLED WATER	NORMAL CARBONATE, CITY WATER	CAUSTIC ALKALINITY, CITY WATER	MAGNESIUM REMOVED	ALUM USED	PROPORTION OF MIXING CHAMBER USED
	Temperature °F.	Total alkalinity						
						per cent	*grains*	
January	32	229	62	42	0	21.6	1.9	Full capacity
February	32	235	62	40	4	32.0	2.0	Full capacity
March	35	171	42	26	9	66.6	2.4	Full capacity
April	47	151	38	26	7	64.7	2.3	Full capacity
May	54	189	42	29	5	52.8	2.1	Full capacity
June	65	197	40	26	7	72.2	1.9	Full capacity
July	73	194	40	40	2	52.4	1.3	Two-thirds capacity
August	72	207	50	39	4	48.1	1.2	Two-sixths capacity
September	63	198	48	38	6	48.7	0.9	Two-thirds capacity
October	48	205	62	40	10	34.9	1.0	Full capacity
November	40	214	58	44	0	32.9	1.2	Full capacity
December	34	234	60	40	1	40.8	1.9	Full capacity
Average	50	202	51	39	5	46.7	1.7	

d. Given some more or less persistent trouble with hot water heating apparatus, gas heaters, furnace coils and the like.

Tables 3 and 4 give in some detail the comparative condition of the sand and the history of the meters for the year 1917–1918.

This represents 17 per cent of the total number of meters in use, 66 per cent of which were lime-coated enough to cause their shopping, an item of considerable expense.

During the winter of 1912–1913, the first year of operation, there was a rather intensified period of trouble with the hot water heating

systems. Steam-coil-heated tanks would become furred 3 to 4 inches thick in a week's time. Instantaneous gas heater coils would become completely choked up, as would the furnace coils, the latter burning out. The author is convinced, however, that much of this trouble could have been avoided by more ample pipe sizes and by the elimination of many unnecessary ells, turns and other reductions,

TABLE 3

Condition of the sand, 1912–1913 and 1918

	SAND AS PLACED, 1912	SAND AS FOUND AUGUST, 1918
Effective size, millimeter	0.27	Above 1.05
Uniformity coefficient	1.61	1±
Acid soluble, per cent	6.5	84.7
Approximate thickness of coating, millimeter	None	0.255
Specific gravity	2.651	2.173
Volume increase, volumes	1	1 sand, 6.5 coatings

TABLE 4

Nature of trouble with 3827 meters sent to the shops in the fiscal year 1917–1918

	PER CENT	NUMBER
No information	4.9	189
Meters all right	0.1	3
Out for test	0.2	7
Sand clogged	1.5	57
Frozen	8.1	310
Mechanical trouble	19.3	738
Lime and mechanical trouble	12.1	461
Limed-up only	53.9	2062
	100	3827

all of which seemed favored places of deposit for the lime coatings. This trouble was the more aggravating in that it occurred at such a time as to make concurrent the most difficult water to treat with the greatest demands for hot-water heating.

Laboratory tests seemed to identify this with the percentage of magnesium removed, such that when the removal effected was 25 per cent or more the trouble from this source was reduced ma-

terially. Operation in the succeeding years, therefore, became controlled by the percentage of magnesium removed, in that with magnesium removed to the extent of 33 to 46 per cent troubles due to after-deposits were apparently diminished and complaints arising from trouble with hot-water hearing apparatus decreased.

Within the last year continued efforts to cope with the small, but persistent, troubles arising from after-deposits, coupled with reasons of an economic nature arising out of the increasing cost of alum, have lead to a change in the author's ideas as to the exact rôle played by the magnesium in lessening such troubles. Formerly, he believed the troubles due almost entirely to colloidal magnesium and bent all efforts towards its removal, through increases of the amount of alum used and the rigid maintenance of residual caustic alkalinity. The fact that magnesium was not a predominating characteristic in the coatings deposited, there being but 8 to 15 per cent therein, did not seem to square with the theory, even though it was a pronounced fact that removal of magnesium was consistent with reduced after-reaction troubles. The author is beginning to suspect that, in reality, the major cause of the more stable effluent lay in a reduced normal carbonate content of the water going to the city mains. While it was true that increase of alum and maintenance of caustic alkalinity increased the amount of magnesium removed proportionately, it also not only reduced the normal carbonate content of the water, but, in addition, maintenance of caustic alkalinity tended to prevent deposit, to some extent, of that portion of normal carbonate that may have been present in a supersaturated state. Table 5 throws light on this.

In 1912–1913 occurred the worst period of after-deposit troubles. For that year the magnesium removal was but 25 per cent and the normal carbonate alkalinity of the city water for that period was greater than for the average of the succeeding years by 51 per cent, being 65 p.p.m. as against an average of 43 p.p.m. for the five succeeding years.

This explains the general nature of the problem that was confronting the author, a solution for which was sought in the experiments discussed in this paper. There was a constant seeking for some method of operation that would compensate for the rather insufficient settling basins for lime softening operations, so that the city would save all the desirable and economic features of a soft water without a continuance of the various troubles discussed and

without the expenditure of increasing amounts of alum, the cost of which was mounting.

The experiments were largely inspired by what was being done with the use of air and sludge in the treatment of sewage and the possible feasibility of pumping back, in like manner, the settling basin sludges into the mixing chamber to intensify the removal of colloids, to save alum if possible, and at the same time to so accelerate the curing of the water as to stabilize the effluent to a point of possibly eliminating entirely the after-reaction troubles and their attendant annoyance to the hot-water user, the expense of cleaning limed meters, and the elimination of sticking gates and valves.

The sludge, in question, is a mixture of some 85 per cent calcium carbonate, 4 per cent magnesium hydrate, aluminum hydrate, mud,

TABLE 5

Average amount of alum applied and removal of magnesium

YEAR	NORMAL CARBONATE, CITY MAINS	CAUSTIC ALKALINITY WATER CITY MAINS	MAGNESIUM REMOVED	ALUM APPLIED
	p. p. m.	*p. p. m.*	*per cent*	*grains*
1913	65	1	25.0	1.0
1914	44	5	33.6	0.7
1915	48	3	37.6	0.9
1916	43	3	43.7	1.2
1917	39	5	46.7	1.7
1918 to August 1	41	6	42.2	1.2

organic matter, etc., and is produced in enormous amounts. It covers the settling basins, which are approximately 110 feet by 210 feet by 12 to 15 feet deep, with a layer of sludge from 1 to 10 feet deep, with one-third to one-half the area covered to a depth of 8 to 10 feet deep, every one and one-half to two months and aggregates 6000 to 7000 tons a year, based upon an average pumpage of 12,200,000 gallons per day and a reduction of the raw water in total hardness, through the use of lime, of from 225 p.p.m. to 95 p.p.m., for the water passing to the city mains, as the average for 6 years, a reduction of 60 per cent.

Preliminary laboratory experiments indicated:

a. That time of contact was not a factor beyond the time required for thorough intermixing.

b. That the volume of sludge in contact was of exceeding importance.

c. That the sludge was most effective when applied to water from the mixing chamber taken at a point just after the alum addition, i.e., at a point where the normal lime reactions were most nearly completed.

d. It seemed most effective, therefore, not so much to hasten the chemical reactions as to remove the products of the reaction.

e. That the sludge did not seem so effective, at least in the volumes attainable by the pumps at hand, when applied to water treated under summer conditions as when applied to water treated under winter conditions.

Table 6 gives results of some of the laboratory experiments and is illustrative of points (a) and (b). The water was taken from

TABLE 6

Results of laboratory experiments in treating water with sludge

SLUDGE ADDED, PER CENT BY VOLUME	TIME	TOTAL ALKALINITY	PH. ALKALINITY	CAUSTIC ALKALINITY	NORMAL CARBONATE	MAGNESIUM
	minutes	*p. p. m.*	*p. p. m.*	*p. p. m.*	*p. p. m.*	*p. p. m.*
0	0	61	28	0	56	16.3
5	3	32	15	0	30	13.4
5	10	29	12	0	24	12.4
10	3	27	12	0	24	11.0
15	3	23	11	0	22	9.6
20	3	20	9	0	18	9.6
25	3	19	9	0	18	8.6

near the end of the mixing chamber after the application of the alum and at a temperature of 32°–35°F.

In each case the amount of air applied was such as just to keep the sludge well up in suspension. The result of this was to form the sludge into large ragged feathery flocs as large as the end of one's little finger that fairly seemed to fall from suspension like sand.

These experiments looked so promising that during the winter of 1916–1917 arrangements were made to carry out a plant-size experiment. At the point of discharge from the mixing chamber into the settling basin a 6-inch centrifugal pump was set up, coupled to a spare 10-horsepower motor. Its suction was dropped into the settling basins at a point where the sludges accumulated to the greatest depth and connected to a line running lengthwise of one section, some 100 to 120 feet long, bored with 1-inch holes on alter-

nate sides of the horizontal diameter and spaced so as to be 1 foot apart on alternate sides of the pipe, i.e., a hole every 2 feet apart on either side. The pump was further arranged to discharge into the mixing chamber at a point some four to six passes from its place of discharge into the settling basin.

Several runs were made with this pump so arranged, the results of which proved largely confirmatory of the preliminary laboratory experiments, especially as regards its relative effectiveness in the

TABLE 7

Results of a typical large-scale test of the effect of sludge; made in January, 1917

TIME	WATER AT GATE K					WATER AT GATE L					SLUDGE USED, PER CENT BY VOLUME
	Total alkalinity	Ph. alkalinity	Caustic alkalinity	Normal carbonate	Magnesium	Total alkalinity	Ph. alkalinity	Caustic alkalinity	Normal carbonate	Magnesium	
	p.p.m.	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	
10.00 a.m.....	100	40	0	80	24.0	66	32	0	64	19.2	0
10.30 a.m.....	98	38	0	76	24.0	74	32	0	64	19.2	0
11.00 a.m.....	103	44	0	88		72	32	0	64	18.2	0
Average.....	100	41	0	81	24.0	71	32	0	64	18.8	0
11.30 a.m.*....	86	32	0	64	22.1	44	20	0	40	13.4	7
12.00 noon.....	70	30	0	60	15.3	46	20	0	40	16.3	6
12.30 p.m.....	94	48	2	92	11.5	46	30	14	32	10.6	5
1.00 p.m.....	136	64	0	128	23.0	64	36	8	56	11.5	3
Average.....	96	43	0	86	17.9	50	26	2	42	12.9	5
1.30 p.m†.....	108	64	20	88	19.2	88	50	12	76	16.6	0
2.00 p.m.....	115	52	0	104	23.0	106	60	14	92	16.6	0
Average.....	111	58	10	96	21.1	97	55	13	84	16.6	0

* Sludge pump started.

† Sludge pump stopped.

winter and summer months and as regards the importance of the volume of sludge in contact with the water under treatment. It proved disappointing in particulars noted further on. The results in table 7 are given as typical of a run made in January, 1917, where the water was followed from the mixing chamber through the settling basins into the clear wells. Note that the water at gate K is at a point one-quarter of the distance from the end of the mixing chamber (at the point found best for the application of the alum) applied on this day at the rate of 1.24 grains per gallon. The water at gate L

is at the point of discharge of the mixing chamber into the settling basins. The sludges were applied at a point midway between gates K and L. During this run, every effort was made to keep the application of the lime as steady as possible, all other factors, as rate of pumpage, being so controlled as to eliminate all variables but the effect of the applied sludge on the water under treatment, since, under the best conditions, the application of the lime is a variable factor having much to do with the efficiency of the final results obtained.

Table 8 is presented to show the average water from gate L for the two periods, without sludge and with sludge, together with the total average of the water at gate K.

TABLE 8

Average results with and without sludge

GATE	PERIOD	TOTAL ALKALINITY	PH. ALKALINITY	CAUSTIC ALKALINITY	NORMAL CARBONATE	MAGNESIUM
		p. p. m.	*p. p. m.*	*p. p. m.*	*p. p. m.*	*p. p. m.*
K	10 a.m.–2 p.m	101	46	2	99	18.0
L	No sludge	81	41	5	76	17.9
L	5 per cent sludge	50	26	2	48	12.9
L	Reduction, percentage	38	37.6		36.6	28.0

It was one of the expectations in connection with such a treatment that with a water so reduced in alkalinity and normal carbonate content there would be a diminution of the drop in alkalinity on passing the filters, which would be strong evidence of the stable character of the water so produced and be strongly in contrast with the usual experience, as noted by comparing the normal carbonate alkalinity of the settled water with that of the filtered water as shown for the year 1917 in table 2, the average reduction of 12 p.p.m. normal carbonate alkalinity explaining entirely the rapid growth on the sand grains. Table 9, therefore, follows the water under study around to the filters before and after passing the sand where it shows but 1 p.p.m. drop as against a 13 p.p.m. drop across the sand for the unsludged water.

The plant-size experiments showed the ability of the sludge treatment to:

a. Reduce the alkalinity and normal carbonate content of the water.

TABLE 9

Change in character of water as it passed through the plant

TIME	SETTLED WATER, SURFACE OF FILTERS					EFFLUENT FROM FILTERS				
	Total alkalinity	Ph. alkalinity	Caustic alkalinity	Normal carbonate	Magnesium	Total alkalinity	Ph. alkalinity	Caustic alkalinity	Normal carbonate	Magnesium
	p.p.m.	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*	*p.p.m.*
11.00 a.m.	84	24	0	48		67	31	0	62	
12.00 noon	64	28	0	56		48	23	0	46	
12.30 p.m.	65	28	0	56		50	25	0	50	
1.00 p.m.	65	30	0	60		53	28	3	50	
*1.30 p.m.	47	21	0	42		45	25	5	40	
2.00 p.m.	40	19	0	38		42	23	4	38	
2.30 p.m.	35	19	3	32		36	21	6	30	
†3.00 p.m.	36	22	8	28	8.6	32	21	10	22	10.5
3.30 p.m.	42	28	14	28	11.5	34	23	12	22	5.7
4.00 p.m.	45	33	21	24		35	25	15	20	
4.30 p.m.	51	35	19	32		40	27	14	26	
5.00 p.m.	55	36	17	38		41	28	15	26	

* At this point the sludge treated water came around on the filters.

† At 3 p.m. the last of the sludge treated water was passing, as can be noted by the return to normal of the alkalinities of the two waters under study, at 3.30 p.m.

TABLE 10

Summary of results detailed in table 9

	TOTAL ALKALINITY	PH. ALKALINITY	CAUSTIC ALKALINITY	NORMAL CARBONATE
	p. p. m.	*p. p. m.*	*p. p. m.*	*p. p. m.*
Average of unsludged, settled water 11.00 to 1.00 and 4.00 to 5.00	61	31	8	53
Average of filter effluents for same period	48	27	1	42
Loss through filters	13			11
Average of sludge-treated, settled water 1.30 to 3.00	39	20	3	36
Average of sludge-treated water after passing filters	39	22	6	33
Loss through filters	0			3

b. Reduce the magnesium content of the water for the winter months beyond that possible without it or else at the possible expenditure of an impossible amount of alum.

c. Produce, with its aid, a water whose normal carbonate alkalinity was reduced to an average of 33 p.p.m., a point 10 p.p.m. lower than that possible over an average of five years as shown in table 5.

d. Produce a water showing a negligible drop across the filters as indicated in table 10, where the drop of from 11 to 13 p.p.m. was reduced to from 0 to 3 p.p.m.

To this extent the process was encouraging, but it proved unfavorable for the following considerations:

a. Because of the difficulty of getting a constant supply of sludge in sufficient amounts. It was assumed that the sludge could be made to travel somewhat as a belt, that it could be lifted into the mixing chamber where it would do its work only to deposit rapidly again, the suctions being laid in a place thought to be most favorable for such abundant redeposit, especially in view of the fact that it was constantly being added to by the newly treated water. Actual pumping, however, soon showed an inability to supply sludge at a rate greater than 5 to 7 per cent by volume of the sludge and for a period of more than one to one and one-half hours at a time, when, in the midst of plenty, the supply would cease entirely.

Sludge could be pumped each time until there had been produced about each suction hole a more or less cleared area, after which the sludge would cease to come and the experiment for the day would have to be terminated until such time as the sludge had again been built up. The sludge absolutely refused to flow to the pump suctions and the permanent adoption of such a scheme seemed dependent on the use of some device that would either bring sludge to the suctions as through the use of a Dorr thickener or else the devising of some type of traveling or flexible suction that would follow up the sludge. Neither of these schemes seems impossible of solution.

b. Because of the production in the water of a finely divided suspended matter, both difficult and expensive to remove by the use of alum, a procedure hardly to be considered in view of the fact that the whole object of the scheme, to conserve the alum, would be defeated. The production of this finely divided suspended matter was at variance with the laboratory experiments and is

somewhat difficult to explain unless the mechanical action on the sludge as it was drawn through the many suction holes and threshed about in the pump impeller were responsible, though there is to be set against this idea the fact that water dipped from the mixing chamber just at its time of discharge showed the same large ragged flocs that were produced in the laboratory with air agitation, and the same tendency to settle rapidly. It was also impossible to get sludge runs long enough to gain an idea of just how such a water would act on the filters either to increase the loss of head rapidly, the amount washwater used, or how it would interfere with the quality of the effluents.

No agitation was used in the plant-size experiments other than that afforded by the normal baffling of the mixing chambers, this point again checking the laboratory findings of the comparative importance of the volume of the sludge used rather than the length of time in contact.

It is difficult to state with sureness any exact explanation of these phenomena. It would appear that they are largely mechanical, in that the sludge used contains no residual chemical activity, it apparently acts to scour out colloidal matter, as evidenced by the reduction of the magnesium content of the water and as further evidenced by the apparently greater efficiency to do so when applied to the water that has its lime reactions most nearly completed, and, finally, as evidenced by the fact that the reduction in the normal and total alkalinity must come about through the introduction of the many crystals of calcium carbonate that act as foci for the more rapid formation of such crystals than normally takes place.

With the establishing of these facts the work was terminated through pressure of other work and the lack of adequate technical assistance, though it is felt that the results attained give sufficient promise to warrant further work thereon. It is understood that a plant at Lawrence, Kan., inspired by these experiments has incorporated something of this idea in its design but all details as to the truth of this or type of design are lacking. Finally the idea seems particularly applicable to the smaller plants, either for the smaller communities cursed with hard well waters, or the intermittent types or the proprietary plant.

DISCUSSION

W. F. MONFORT: The principle of the scheme is undoubtedly sound; to prolong the period of contact or increase the area offered by the sludge as sand in the sand filters coated with calcium carbonate and mixed with magnesium compounds assists in this reaction. At Quincy, Ill., no incrustation occurred in the distribution systems but the sand had less than 25 per cent of sand and over 75 per cent of other incrusting matter.

M. F. STEIN: As regards the deposits that the author said did not contain an extensive amount of magnesium, the same thing occurred in Cleveland with plates used in testing softened water. The deposits in the case of plain iron plates were composed largely of carbonate of iron; in the case of galvanized iron, carbonate of zinc; and in the case of brass plates there was a very slight deposit, which was rather surprising owing to the trouble that is often caused with lime-softened water coating valve seats. There was also practically no deposit on glass plates, which is rather interesting from a theoretical view point, as glass is an ideal surface for contact action in colloidal phenomena. With magnesium in colloidal state, one would expect it to deposit on the glass and be coagulated by the glass through contact action.

A question suggesting itself regarding the continued use of sludge is whether the sludge will keep in a sufficiently fresh condition. At Cleveland it was found that the deposit of sludge in the settling basin, over perhaps ten days or two weeks old, would develop a very strong odor which was highly disagreeable and made necessary the cleaning of the basins. It is not known whether this was due to the lime and magnesium salts or whether it was due to the organic matter in the water.

W. A. SPERRY: One thing that interested the author at Grand Rapids at the starting of the plant was the similarity of the sludge produced to that produced at Columbus, Ohio, where lime softening is accomplished in a similar manner. When cleaning the basins the fishy odor mentioned by Mr. Stein was strongly pronounced and so similar to the odor of the Columbus sludge during cleaning there, that if one were taken blindfolded into either plant one could not tell which plant was visited. Apparently while pumping the sludge back, although there was evidence of this odor being present as the

sludge was brought to the surface, it did not transmit itself to the water in amounts sufficient to be detected.

The amount of sludge produced varies with the amount of hardness carried by the water as affected by the varying seasons, amount of rainfall, etc. During the winter, with the rivers largely fed by springs, the water increases greatly in hardness, with a corresponding increase in hardness to be removed. With the passing of the winter's snow and spring rains, there is a diluting effect, reducing the amount of softening necessary and the sludge produced as a consequence, but also greatly increasing the suspended clay, silts, sand, etc. The figures given in the paper are very good averages for the year where the sludge produced from an average dosage of 8 to 10 grains of 88 per cent lime per gallon will amount to from 16 to 20 grains per gallon settling out in the settling basins.

TYPHOID FEVER AT MOLINE, ILLINOIS[1]

By M. C. Sjoblom

During the fall of 1917, reports received by the State Department of Public Health indicated that an unwarranted amount of typhoid fever existed in Moline. This state of affairs continued and the author was detailed to investigate it. A study of the conditions between September 1, 1917, and April 1, 1918, showed conclusively the source of infection to be city water. Recommendations were made for safeguarding the water supply against further infection. In July, 1918, the Department received reports indicating an explosive outbreak of typhoid. A second investigation showed conclusively that the city water supply was again the agent of infection. Following closely the main outbreak in July a small local outbreak occurred which was traced to water from a well, doubtless infected through near-by sewers.

Moline, a manufacturing city of somewhat over 30,000 population, is situated on Mississippi River. Moline, Rock Island, and East Moline comprise one community. Rock Island has a population about the same as Moline. East Moline has had a very rapid growth and probably has in the neighborhood of 8000 inhabitants. During the three years prior to the opening of the filter plant in 1902, the annual typhoid death rate of Moline was 100, 144, and 151 per 100,000; since then it has been 22 to 60, except in 1918, when it jumped to 80 in the first eight months.

The cases in the last outbreak were found in every part of the town. The general distribution of cases throughout Moline, as well as the fact that Rock Island and East Moline had but few cases and the majority of these gave histories of having received infection in Moline, pointed quite strongly from the beginning toward the water supply as the source of infection.

There was a preponderance of male cases, accounted for by war conditions causing local factories to offer attractive wages, bringing

[1] Read before the Illinois Section on March 25, 1919.

a large number of men to Moline for employment. The majority of these were young men without families.

The main factory district lies along Mississippi River. In order to receive minimum insurance rates from the underwriters, it has been necessary for the factory owners to provide two sources of supply for fire purposes. To meet this demand, what is known as the manufacturers' main has been laid between the factories and the river. This main is connected with the city supply. There were but few connections made originally, but as time went on additional connections were made, until 20 or more are now in use. The manufacturers' main is provided with single check valves near the points at which it is connected to the city mains. Single check valves are also provided on all cross-connections. By-passes equipped with small gate valves are placed at some of these points.

The check valves are installed to open toward the factory side of the connections, thus permitting the city water to enter the system, but tending to prevent a flow into the city mains should a breakdown or insufficient pressure in the city supply make it advisable or necessary for the factory pumps to be operated. The city pressure is ordinarily carried at about 110 pounds.

As now operated, the city supplies quantities of water to only a few factories. A number have their own pumps forcing river water into sections of the factory pipes, which are presumedly shut off from the city. However, the factories have fire pumps installed for emergency purposes and these are tested weekly or thereabouts to see that they are in good operating condition. Annual fire tests are also made, which in some cases are quite extensive, but during these periods all precautions are presumably taken to prevent the water from entering the city mains.

In the recent outbreak the attention of many of the citizens was at once centered on this factory water pipe arrangement. As a result, the city officials took it upon themselves to test out one or more of the connections to see if the check valves operated properly. Accordingly, one Sunday they closed the proper gate valves on the distribution system of the city which would permit water to enter a section of the city mains between these valves should the checks leak. It was stated that upon opening a hydrant in this section of pipe, the water came through the check with sufficient force to be thrown to the opposite side of the street. The valve here tested was one which had been installed for a large number of years and it was

not at all surprising that it leaked badly. There is not a check valve made which can be depended upon to remain water-tight unless systematically cared for and it is safe to predict that had tests been made on all the checks the majority would have been found to leak.

Considerable sewage finds its way into the river at relatively short distances above the Moline water works. Some of these towns discharging sewage above Moline had more or less typhoid. For instance, Clinton, Iowa, about 35 miles upstream, had a number of cases during the fall of 1917 when Moline was also suffering from the disease. East Moline and Watertown Hospital, both of which had a number of cases of typhoid, discharge sewage about 5½ miles above the water works. In addition, a sewer serving the east end of Moline discharges above the intakes. Though these conditions do not prove the exact point from which the infection entered the river, they indicate the dangers with which Moline had to contend, and the need to operate the water-purification plant efficiently at all times. It is quite likely that the sewage from some of these points actually constituted the sources of the infection which entered the city mains and caused the typhoid.

During the outbreak of typhoid in the fall of 1917, the thickness of the filtering medium in the water treatment plant had been reduced about 10 inches below the original thickness. A large part of the underdrain system had become clogged, making it necessary to maintain excessive rates of filtration in portions of the bed in order to supply the demands of the city. With the low efficiency of the filters in this condition the quality of water was questionable. Ordinarily no bad result would have followed, as liquid chlorine was fed into the effluent as it passed to the clear wall, but the records showed that the chlorinator refused to operate on a number of occasions and as a result poorly filtered water was pumped into the mains. This may have been responsible for a number of infections. But the main number of cases doubtless resulted from another factor in the plant, a by-pass. This was not given due prominence during the first investigation, but information secured in a later investigation made it quite apparent that it played a major rôle in the typhoid infection.

Some years ago the commissioner of water works decided to make it possible to pass settled water into the clear well at such times as the demands were greater than the filter capacity. This often occurred after the strainers began to clog, and when the clear well,

which had a very limited capacity, had been heavily drawn from. In order to carry out this idea a gate valve was placed in the pipe leading to the sewer from two wash troughs adjoining a wall common to two filters. By opening the valves leading from the troughs to this pipe and also opening the valve used if filtered water was to be wasted into the sewer, an open channel was provided through which the settled water passing over these two filters could enter the clear well. Then by closing what was known as the by-pass valve, which was installed below the point at which the waste line was connected with the sewer line, the water was forced to pass through the waste line into the clear well with the filtered water.

The records indicated some of the dates on which the by-pass was opened during the fall of 1917, but there is reason to believe that water had been by-passed on other occasions and that the chlorine had been off during the same period.

Following the outbreak in the fall and winter of 1917 and 1918 the city had all the filters overhauled, the defective strainers replaced, and better sand used. Orders were also given for two chlorinators to replace the machine which had for one reason or another given endless trouble. With the filters overhauled and the city alive to the dangers of permitting a partially treated water to enter the city mains, it was thought that no further trouble would be had. However, in July a sudden and serious outbreak of typhoid fever occurred.

This outbreak, like the preceding one, comprised cases scattered throughout the city. The same method of eliminating possible sources of infection was used as in the former investigation, with the result that all evidence pointed conclusively to the city water supply. With the filters newly overhauled, it was evident that the sudden and extensive outbreak could not have occurred from filtered water, even had no chlorine been added to the effluent. There was reason, therefore, to believe that the by-pass had been used.

The plant records showed that the old chlorinator had given considerable trouble and a new chlorinator had not been installed until July 1, which was after the infection must have occurred but, as before stated, this alone could not explain the cause of the outbreak. There still remains the possibility of the by-pass being opened, but the records showed no occasion where this had occurred which could be responsible for the main outbreak. The chemist was, therefore, questioned as to the possibility of the by-pass having been opened but not recorded. No instance was learned, the chemist

stating that the filters in their existing condition were capable of yielding sufficient water for all needs and the by-pass had not been used. In spite of these denials, the evidence appeared so conclusive against the city plant that it was decided to simply remain at the plant on the theory that some one would eventually disclose some incident which would lead to a solution.

As expected the necessary information was secured from an employee. The chemist had opened the by-pass about 5 p.m. on June 21 and left it open until about 2 p.m. on June 22. During this period one of the sedimentation tanks had been cleaned, cutting the sedimentation period in two, which permitted water even poorer than usual to enter the clear well. A study of the records showed that the chlorine was on only a part of the time the water was being by-passed. Later information was secured from a diary kept by the chemist that the by-pass was also open during a part of June 19. During portions of this period little or no chlorine was being added. There is no doubt but that the typhoid outbreak in July resulted from the opening of the by-pass on the dates mentioned.

Measures were again taken to prevent any more infectious matter entering, but more cases were reported than should have resulted from secondary infections. It was soon found that the majority of these cases lived in a certain section of the city or else had been working or visiting near-by. A careful study made here indicated that a private well was responsible for a second outbreak in which perhaps 25 cases became infected. This well is of the dug type, approximately 75 feet in depth. About 72 feet of this depth is in limestone. The top of the well was adequately protected. The exact channel through which this well became infected is not definitely known, but there is every reason to believe that the sewage from an adjoining building which had contained some typhoid cases may have entered the well and caused the trouble, or that sewage from sewers in streets adjoining, which are laid in the limestone formation, may have travelled through a crevice in the rock to the well.

The conclusions to be drawn from these incidents are:

1. In a filter plant treating a polluted stream water by-passes should under no conditions be tolerated. This means that the plant must be of ample capacity to supply sufficient filtered water at all times.

2. Dual connections, such as exist at Moline, should not be permitted. Although in this case nothing indicated that the raw river water from the factory main entered the city supply, still a real danger exists under these conditions. Even should double checks be installed, the personal element in caring for these valves is a source of danger.

3. Records ought to be kept of all operations, even if there is danger from criticism being raised later relative to certain acts. Nothing will help more to reveal dangerous conditions than to note every operation, so that if some trouble arises the full records can later be studied. If this had been done at Moline, the cause of the outbreaks would have been very easily found. Often what appear to be trivial incidents to the average attendant may prove of great importance when studied in connection with other facts.

4. There is a real danger in using a water supply from a limestone formation and especially where the well is located near sewers or privies entering the limestone. Another case in Illinois has again brought out this fact.

DISCUSSION

H. E. KEELER: There is no question there was a typhoid outbreak at Moline, in fact the city chemist has, in his paper on the filtration plant, stated a second outbreak occurred. The thorough investigation made by the State Board of Health clearly showed that among other possible causes for the outbreak were the numerous connections between the mains of the city water service and the private fire supply services installed in large manufacturing plants which take their supply from the river close to the shore line and depend on keeping an impure supply from entering the city mains by the use of ordinary check valves, many of which were installed for a long time. The author referred to a test made on one of those connections which showed that the leakage was so great that water was thrown from a hydrant across the street showing that practically it was of no service as a tight valve. This may have been an isolated and aggravated case, but it showed what might happen to any such connection so made and connected by the use of the wrong type or an inefficient check valve. Therefore one of the conclusions in the paper is that the use of check valves would be unsafe.

This would be a good conclusion to arrive at, if it was a fact. The fact is, there are efficient and safe check valves made, designed for

the purpose of preventing an impure supply getting into a pure city supply. The great necessity for such a device was brought out several years ago by reason of a typhoid epidemic that occurred in a prominent manufacturing city in the east. A careful investigation was made to ascertain the causes. Finally it was found that the city water supply was contaminated by impure water reaching the mains through fire service connections in several of the factories, which were not fitted with efficient, tight valves. The result was that the underwriters, health authorities, and several of the manufacturers of these appliances got together, and while the problem was a big one, it was solved and an appliance was designed, tested and approved by the underwriters, health authorities and the American Water Works Association. It has been successfully installed and, as far as the speaker knows, it has never failed to serve the purpose for which it was designed. This apparatus is made by several valve manufacturers. It consists of a specially designed, flanged, straight-way bronze-mounted check valve, with rubber-faced clappers and extra large clearances, designed to resist corrosion and to prevent derangement of the parts. They are installed in pairs and arranged with test valves and gauges so that they can be tested and inspected frequently for tightness and efficiency. They are usually tested quarterly, but of course can be tested more frequently if desired. By using such an apparatus, properly installed, the risk of danger from an impure supply is practically eliminated. Their cost is of course quite a bit higher than for regular or even underwriter check valves, but when human life is at stake, cost is quite a secondary consideration. The speaker has never known it to be considered by large manufacturing concerns which have occasion to meet such a situation.

A. E. ANDERSON: The typhoid rate, as stated by the author, has been high at times other than the mentioned epidemics, and as pointed out by him some 25 cases in the epidemic mentioned were due to a well in the western part of the city. This, and the fact that the city is fairly loaded down with privies and analyses of most well waters show them to be contaminated, is the reason for the speaker's opinion that this high rate may be largely attributable to wells and privies. In some cases the stools from typhoid patients have been dumped into these privies without any use of germicides and in other cases the germicidal material has been improperly used.

M. C. SJOBLOM: If the contamination had come from leaky check valves, the epidemic would have been more local. The pressure in the city mains is usually about 110 pounds, while the pressure in the factory fire system is never raised much above 80 pounds, so that it would be only during a breakdown or drop in the city pressure that factory water could enter the city mains, even should the valves leak.

H. P. T. MATTE: It is desirable to amend Mr. Keeler's statement regarding the use of a number of check valves by the Moline industrial plants. These will be installed as an additional safeguard only, because at all times pure city water is to be used in the factory lines. In cases of a conflagration, however, and if the city pumping plant should fail, the check valves will permit the factories to fight the fire with raw river water without contaminating the filtered water in the city mains. The industrial plants are willing to pay the price of the valves for an additional protection, both against fire and against disease.

W. W. DEBERARD: The fact that impressed the speaker was that after filtration was begun typhoid dropped down at times to 20 to 25, but that is still high. Mr. Anderson has given a good reason for that, namely, that there are a large number of wells in a poor state of repair and little attempt has been made at disinfection in typhoid cases.

C. K. CALVERT: There is a protecting scheme to be had which any company can install on services of consumers who have an impure supply which may be forced back into the public mains. On the service line three valves may be placed, the middle one a check valve. Between it and each of the end valves a small tap in the main shows on inspection whether or not the check is operating. Inspections should be made very frequently, since failure to operate may produce serious results. This is an almost perfect protection.

THE OPERATION OF THE MOLINE FILTRATION PLANT[1]

By A. E. Anderson

This plant is municipal property, and is situated on the bank of the Mississippi River. It is 242 feet in length and 76 feet deep, faces the river on the north and supplies a city of 35,000 inhabitants. It has a Jewell system of sedimentation and filtration, a pumping capacity of 20,000,000 gallons per twenty-four hours, filtering capacity of 5,000,000 to 6,000,000 gallons, and reserve of 1,000,000 gallons. The western portion of building is occupied by the boiler and locker rooms, the central portion by engine and tool rooms, and the eastern portion by sedimentation and filtration room, laboratory, meter room, alum storage.

Boiler room. This is equipped with two Aultman-Taylor Illinois stoker feed boilers of 150 horse power each and two Keeler boilers of 200 horse power each, hand stoked and fitted with Palmer forced draft. There are two Worthington duplex boiler pumps and one Blake & Knowles 1,000-horse power open heater. The water is heated as a rule to 190° Fahr.

Raw water supply. The water is obtained from the two channels of river, which branches on account of a dam extending $2\frac{1}{2}$ miles upstream from the eastern point of the Government Arsenal, an island directly opposite Moline. This dam was built to increase the head 5 feet between the Arsenal and Moline in order to supply water power for the Arsenal and local electric plants situated a mile downstream.

Intakes. These are two in number, one 18 inch extending 3280 feet northeast to the main river channel, the other a 20 inch extending 1200 feet due north to the channel of the water-power branch. The water enters the plant by gravity. These intakes are equipped with a compressed air connection to blow away any obstruction such as ice which may threaten to shut off the supply.

Raw water reservoir. This is located beneath the floor in the northwest corner of the engine room. It has a depth of 20 feet and a

[1]Read before the Illinois Section on March 25, 1919.

capacity of 90,000 gallons. The head of water here varies with the river stage because of its flowing in by gravity.

Low-lift pumps. These are two in number and lift the water to basins 75 feet to the east. The lift varies from 16 to 26 feet, depending on the river stage.

One of these pumps is situated in the northeast corner of the engine room, half way between the raw-water reservoir and the basins. It is a single-stage, centrifugal pump having a capacity of 5,000,000 gallons per twenty-four hours. It is driven by a double-wheel water motor, the actuating water being taken off the high-service main at 100 to 110 pounds. This water flows back to the clear-water reservoir.

The other pump is an auxiliary used only at times of heavy demands. It is a Worthington double-stage centrifugal pump driven by a Kerr steam turbine, and has a capacity of 6,000,000 gallons per twenty-four hours. It is located in a pit 12 feet below the engine room floor and 4 feet east of the raw-water reservoir, making a suction lift of 0-8 feet; at high water there is practically no lift. The first-mentioned pump has a suction lift of 12 to 18 feet. This Worthington pump is connected to the 18 inch main and can pull directly from the main channel or reservoir as desired.

Alum introduction. The alum can be introduced in any one of three ways, by pump, suction or siphon. The pump discharges the alum into the raw water main at a point 50 feet from the basins. The suction line is attached to the suction lifts of both low-lift pumps. The siphon will discharge the alum charge at the same point as the pump or at the weir where the water flows over to the filters.

Alum tanks. There are two of these, from which the charge is pumped, suctioned or siphoned. They are concrete, 12 feet in diameter, 10 feet deep, and located in the southwest corner of the filtration room. The alum is dumped into a solution tank located between the two main tanks. The water is turned in at the bottom of the solution tank and the charged solution flows through a strainer at the top into either tank. The latter are equipped with float gauges.

Alum pumps. There are two alum pumps, each having a capacity of 40,000 gallons per 24 hours. They are water-motor, centrifugal pumps operated by pressure from the high-service main, the water used flowing back to the clear-water reservoir. These pumps are

located beneath the floor 50 feet north of the alum tanks and 15 feet east of the point of alum introduction.

Suction lines. These are beneath the floor and, as above stated, discharge alum solution into the suction lines of both low-lift pumps.

Siphon. This is an emergency means of supplying the alum. It is used chiefly by operators to increase the floc on the filters, to insure having a sufficient mat on the beds, when for any reason the floc is coming over the weir to the filters in insufficient amounts. This siphon can also be used in case of emergency for hypochlorite of lime treatment.

Basins. The water, after being dosed with alum, enters two basins, separated by a concrete wall to permit washing one at a time. They are 50 feet long, 32 feet wide and 20 feet deep and extend 8 feet above the floor level. Both the basins and filters are surrounded by a railed walk. Each basin is subdivided into five smaller ones, each 10 feet wide and 32 feet long. These divisions are made by wooden baffles, which have 6 x 24-inch openings 6 inches apart from top to bottom. The baffles, having these spaces at opposite ends, cause the water to take a zigzag course. Each basin has a capacity of 240,000 gallons. The average period of sedimentation is one and one-half hours. The basins are washed every month, treated water being used.

Weir. After sedimentation in these basins the water flows over a weir in the last sedimentation basin into a 24-inch main which feeds all filters. Each filter has a 10-inch connection at the center, which branches to an 8-inch extending to the sides of the filter, with the exception of filters 1 and 5, where the connection extends but to one side.

Filters. These are located directly back of the settling basins. They are five in number, $12\frac{1}{2}$ feet wide, 33 feet long and 9 feet deep. Running through the center lengthwise at the bottom is a 12-inch main with $1\frac{1}{2}$-inch pipes extending at right angles to each side of the filter. These are placed 6 inches apart the full length of filter. On these $1\frac{1}{2}$-inch pipes, at a distance of 4 inches apart, are Jewell Pittsburg strainers. Directly above this center main at the bottom and 25 inches above the sand level is an 8-inch air pipe having eight 2-inch connections extending down to the bottom main. Through this the filters are given an air blow, usually three to five minutes three times a week.

The filters have 12 inches of graded gravel and 30 inches of Red Wing sand.

The wash troughs for each filter are two in number, located at the sides. They are 12 inches wide and 20 inches deep.

The wash water used is pumped from the clear-water reservoir by a 60-horse-power Erie engine, aided at the present time by some pressure from the high service main. There is a rise of 14 inches per minute through the sand and it is customary to use about 30,000 gallons per ten minutes. The controllers are fitted with two discs, a large disc giving a flow of 1,000,000 gallons for 24 hours and a smaller disc allowing 200,000 gallons more per filter or a total for the five filters of 1,000,000 gallons more per 24 hours. Experiments are now being carried out to ascertain if there is any appreciable difference in the quality of the filtered product.

Float loss-of-head gauges used. When starting a filter, the water is allowed to stand quiescent on the bed for 15 to 30 minutes, and the effluent is then run to waste for 5 minutes, in order to bring the mat down firmly on the sand before water is turned into the clear-water basin.

Controllers. These are the Jewell four-valve float type and, as already stated, have two discs, the large telescoped over the small, thus permitting a change in five minutes to either disc desired.

Chlorination. The water after leaving the filters passes into a 24-inch main which extends north 15 feet to the clear-water reservoir, entering at the bottom. Just before entering, it is chlorinated.

A Wallace & Tierman manual-control solution feed, type B apparatus has been used; an automatic apparatus is now being installed. This apparatus is located in the northwest corner of the sedimentation and filtration room, 50 feet from the point of discharge of the chlorine into the water.

Clear-water reservoir. This is circular and covered. It is 90 feet in diameter and 20 feet deep, with a capacity of 700,000 gallons. It has a baffle wall 9 feet high to permit cleaning half the reservoir at a time. This cleaning is accomplished by using a siphon, as the reservoir is below river level.

Pumps. The high-service pumps which force the filtered water into the city mains are a 10,000,000-gallon Epping Carpenter used continually, a 6,000,000-gallon Holly for emergency use, and a 5,000,000-gallon Dean for emergency use. The Epping Carpenter pump has a 30-inch suction, while the Holly and Dean pumps use a common, 24-inch suction.

Venturi. All water leaving the plant is measured through a Venturi meter, 24 inches in diameter with a 10-inch throat.

Standpipe. The pressure in the city mains is 100 to 110 pounds, this being necessary to boost the water into a standpipe 1¼ miles from the plant. This standpipe is 118 feet high and is situated on a bluff 161 feet above the level of water works plant. It has a capacity of 500,000 gallons. It floats on the main all the time and is equipped with an electric gauge which has eliminated all danger of overflowing. The gauge is located at the plant and the mechanism at the standpipe; it indicates the quantity of water in the tank at all times. It has now been in satisfactory service for two years. It was designed by S. E. Buck, engineer at the plant, and F. H. Harrington, city electrician.

By-pass. The use of the by-pass does not mean that raw untreated water is sent into the mains, but that the river water goes through the regular alum treatment and sedimentation, skips the filters, is chlorinated, and is then passed into the clear-water reservoir.

Filters 3 and 4 have their sewer valves opened, allowing the water to flow over the weir of the sedimentation basin into the wash troughs and through the main leading to the sewer, but by opening the by-pass valve the water is turned into the regular main leading to the clear-water reservoir, on the way to which it is chlorinated.

Condition of plant. This plant has a number of problems to be worked out but the author has experienced no particular trouble in keeping up either the quantity or quality of the effluent. The water conforms to the U. S. Treasury Standard during most of the time and a fire of January 15, 1919, showed that the plant can meet any emergency demand up to the limits of its capacity. This fire lasted four hours; Venturi readings showed a pumping of 2,250,000 gallons and there was a reserve supply of water when the emergency was over.

DIFFERENTIA OF B. COLI[1]

By Everett Judson

In the 1917 edition of the American Public Health Association's *Standard Methods of Water Analysis*, it is recommended that the B. coli group be considered as including all non-spore forming bacteria which ferment lactose with gas formation and grow aerobically on standard solid media.

It is the purpose of this paper to consider briefly the two sub-groups of the B. coli group which are of the most importance to the water bacteriologist, namely, B. coli communis and B. lactis aerogenes.

In 1893 Theobald Smith published his well-known paper, "The Fermentation Tube with Special Reference to Anaerobiosis and Gas Production among Bacteria," in which he described the fermentation tube originated by him. In this paper he gave the results of his studies of gas production by certain bacteria, and suggested that the B. coli group might be divided into two distinct sub-groups by reason of the action of the organisms upon saccharose. In the years intervening between the presentation of this paper and the present, we find a number of investigators reporting work done on gas formation and gas ratios. In the light of our present knowledge of the subject we must attribute their varied conclusions to the method and degree of accuracy used in their work.

In 1914 and 1915 Rogers, Clark and Davis, and Rogers, Clark and Evans found that the accurate determination of gas volumes and gas ratios produced in the anaerobic fermentation of glucose furnished most valuable data. The work of Rogers, Clark, and Lubs, 1917, on "The Characteristics of Bacteria of the Colon Type Occurring in Human Feces," agrees with these former conclusions, and gives a new working basis for the study of the B. coli group. Later in the same year Rogers, in his paper, "The Occurrence of Different Types of the Colon-Aerogenes Group in Water," draws the conclusion that there are two types of the colon-aerogenes group which occur in fecal matter in large numbers.

[1] Read before the Illinois Section on March 26, 1919.

By determining the ratio of the carbon dioxide gas to the hydrogen gas produced in anaerobic fermentation of glucose by the B. coli group, we can divide this group into two distinct sub-groups. The one having a ratio of about unity is designated B. coli communis or low-ratio sub-group, while the other, having a ratio of 1.5 or higher, is designated B. aerogenes or high-ratio sub-group.

It has been shown by these more recent investigations that the final hydrogen ion concentration of a given media correlates perfectly with the gas ratios, and by the use of methyl red as an indicator we can readily make a colorimetric differentiation between the low and high ratio sub-groups (low ratio, B. coli, methyl red positive; high ratio, B. aerogenes, methyl red negative).

In routine work we deal with colonies fished from endos media which has been streaked from lactose broth tubes after twenty-four or forty-eight hours incubation at 37°C. The growth on agar slants made from these colonies is inoculated into the methyl red media and back into lactose broth. The gelatin liquefaction test may also be made to eliminate cloacae. The standard methods of water analysis, with the corrections as suggested by Hasseltine, are followed by the author. It is then generally assumed that the organisms of the methyl red positive sub-group are of fecal origin, while those of the methyl red negative may be from either fecal or other origin.

If the low-ratio or B. coli communis sub-group is alone shown to be present it fixes definitely the source of the pollution.

If the high-ratio or B. lactis aerogenes is alone shown to be present the pollution may come from either fecal or non-fecal origin.

Rogers, Clark and Lubs suggested means of differentiating lactis aerogenes of fecal and non-fecal origin. Whether or not this differentiation will prove satisfactory in routine work can only be decided after repeated trials.

In some recent work the author collected water samples from sources exposed to varying degrees of pollution and isolated typical and atypical colon colonies on endos media. Inoculating tubes of methyl red media with 133 colonies, it was found that 8 atypical colonies gave the methyl red positive result while 7 typical colonies gave a negative result. This one example shows the failure of endos media to differentiate the colon-aerogenes group. The opinion is held that many bacteriologists attribute to endos media a differential action for which it was not intended and does not perform.

It is believed that wherever possible the water analyst should differentiate between B. coli communis and B. lactis aerogenes by use of the methyl red test. A sufficient number of colonies should be isolated from each sample tested to determine the relative quantity of the organisms present.

This procedure will bring us one step nearer our ultimate aim: the determination in the laboratory of the exact nature and source of the polluting organisms with which we have to deal, in order that laboratory work may supplement and substantiate the field survey and enable the executive authorities whose duty it is to eliminate the source of pollution to act with intelligence and confidence.

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DISCUSSION

W. F. MONFORT: What conclusion may be drawn from the adonite reaction? Suppose the organism is fecal B. aerogenes, what should be done about it; is the water safe?

EVERETT JUDSON: If the organism is fecal aerogenes, it would be incorrect to say the water is all right.

W. F. MONFORT: Are we to accept the adonite reaction as discriminating between fecal and non-fecal aerogenes? It seems that there is room for work to be done before we accept this as conclusive.

EVERETT JUDSON: It just opens the field. It suggests a new line of investigation. We should not take the work done up to date as final at all; the more experiments, the better we can draw conclusions.

W. F. MONFORT: The present difficulty with adonite seems twofold; getting the material and interpreting results. It may be said that some work has been done here with adonite and with other sugars to get discrimination between varieties and the relative number of B. coli and B. aerogenes. Incidentally there are several kinds of spore-forming organisms and other organisms fermenting lactose which do not conform to the tentative classification in the 1917 Standard Methods.

M. F. STEIN: The quantitative interpretation of the B. coli test deserves attention. Usually the results are stated as a percentage of positive tests; that is, a certain number or percentage of the fermentation tubes shows gas. Now, of course, it would be much better to have the actual number of coli directly. An approach to that has been made in Professor Phelp's coli index. Some time back the speaker gave the matter some thought, and tried to arrive at a numerical interpretation of the coli fermentation test by means of the theory of probability and chance. Taking a sample of water you can divide it, in thought, at least, into a large number of very small droplets, and consider these from the viewpoint of the black and white balls that you read about in the theory of probability. That is, consider the coli as the black balls and the particles of water as white balls and your test then consists of drawing a certain number of these black and white balls from the water sample. Without going into details of the development, it can be shown by this method that the most probable number of B. coli corresponding to any percentage of positive tests is equal to the hyperbolic logarithm of the percentage of negative tests. That is, calling x the number of coli, this is equal to the log Q, Q being the percentage of negative tests. By using this method, the interpretation of coli tests is more logical and more easily understood.

C. K. CALVERT: There is quite a little written on the matter of overgrowth. Is it proper to fish from an endo plate made from one tube more than one organism, or rather is it proper to enrich such portion of water that it may contain more than one gas-forming

organism? There is no assurance that the numerical relationship between types is preserved in enrichment, and by overgrowth the colonies chosen may all be of one type. Would it not be better to plant several portions of water of such volume that some contain gas formers and some do not? This gives reasonable assurance that in portions forming gas only one fermenter is present. It is the speaker's practice to plant ten portions of each of two or three volumes, based on past findings of B. coli concentration in the waters under examination. This is believed to give very superior information. The reason the speaker gave up adonite is that he found it to be a determining factor in too small a number of cases to make its use worth while.

EVERETT JUDSON: We will have to standardize our media, P_h values, quantities and technique, before we start to worry about whether our interpretation is as well made as might be. There are many errors in our present procedure which throw us out more than the errors we make in the interpretation of results.

M. F. STEIN: Regarding the remarks just made, it is a good idea to have an ideal to work to, and an exact interpretation, even with present errors involved, will help us to correct errors. In plotting up a large number of results obtained from different filtration plants, the speaker used this formula and believes he is on the track of at least one important source of error which so far has escaped notice, simply by plotting his ideal log and then plotting the actual results and comparing them with the ideal curve. There is a general tendency for them to differ in a certain way, which, if traced out, may bring to light an important source of error which has not so far been considered.

W. W. DEBERARD: If there are a great number of samples there may be something in it. We might find some errors of judgment have been made in tests of filtration plants in a long series of samples. It is difficult to see how it will help in sewage disposal or in the case of a stream where the water changes vary rapidly.

WATER TESTING STATIONS[1]

By W. T. McClenahan and R. S. Rankin

The development of water purification methods has been accomplished principally with the aid of water-testing stations. An engineer in designing a water purification plant prefers to follow where possible, the indications afforded by the experience of others. Frequently, however, he has no safe guide and recourse is best had to actual tests on a working scale as nearly as possible comparable to real conditions. The critical factors in the design can thus be determined. These may be hydraulic values, such as velocities, settling periods, rates of application; biological values, such as bacterial removal; or physical values, such as the removal of tastes and odors. Briefly, the development of the testing station has been from the general to the specific. The early stations endeavored to formulate fundamental principles which have been applicable to a certain extent to all filtration plants. With these principles established, the testing station has been applied to more detailed and specific problems.

DESIGN AND CONSTRUCTION

The size of the testing station which it is advisable to build depends on the nature of the tests and the funds available. In general, a testing station is a miniature replica of a complete purification plant. Sometimes, however, the tests are run on parts of a completed plant prior to enlargement, or a portion of one of the component parts of a plant is built in advance for test purposes. When it is necessary to determine only the rates and areas required to make established basic principles applicable to local conditions, this method of testing would seem to offer great possibilities, first, because full-size units can be used for the tests, and, second, because the testing station can very often be built out of funds already appropriated for partial or complete treatment.

[1] Read before the Illinois Section on March 26, 1919.

When a small testing station is to be built, the purpose of the tests should be kept clearly in mind. A test run to investigate color removal, for instance, needs only to prove that it is possible to remove color. It is not necessary to produce absolutely sterile water in the testing station itself, for experience has shown that it is much more difficult to get sterile water from a small test filter than from the larger units used in regular filtration work. As long as the test demonstrates that the color can be removed and shows how it is to be done the plant has accomplished its mission.

A testing station should be compact, convenient, and easy of access to all parts. As far as practicable the course of the water through the plant should be progressive. Each step in the treatment process should be in logical sequence, in order that the work of the operator may be simplified and that it may be easier to watch and understand. This is particularly important when the operator is a non-technical man.

The designer should keep in mind that a plant of this kind is often subjected to radical alterations. The baffling and piping arrangements are most often affected, and the chemical used for treating purposes may have to be changed also. In this case, a change in the method of applying or in its point of application may be required.

Because of the temporary character of a testing station, the first cost is relatively very important, while durability of material and cost of operation are not so important. Wooden construction is quite suitable, sheet iron is useful and black iron pipe is often satisfactory. The salvage value after the plant is dismantled may, however, call for better material than would otherwise be used. Obviously, the material and construction should be substantial and good enough to give satisfactory service and to last throughout the period of test.

Recently, the firm of Pearse and Greeley have designed and built two small testing stations to study unusual problems in water purification. They show the value of such tests when little literature or experience is available.

WHITING TESTING STATION

Whiting, a city of 8200 people, is located along the south shore of Lake Michigan in the heart of a great industrial district. Sur-

rounding it are Indiana Harbor, East Chicago and Hammond. Not far distant lie South Chicago and Gary. At present all the sewage from this great manufacturing region finds its way into Lake Michigan, but the Calumet Sag Canal promises some relief in the near future. Some of the industries produce large quantities of troublesome wastes, notably several chemical works, a glucose factory, and the great steel mills. The most troublesome waste, however, is that from the Standard Oil refinery located in Whiting, which imparts a very objectionable taste and odor to the city water. The amount of oil actually existing in the water is exceedingly small and is very difficult to detect by any chemical means, nevertheless the effect is quite apparent to the consumer.

Most of the oil is said to come from the agitators where motor spirits are clarified. Soda and sulphuric acid are added in this refining process and the resulting sulphates are washed out into the lake and form what is called "soap" or "white water." This white water carries off small quantities of oil in mechanical suspension, some in the form of an emulsion and some in colloidal solution. Portions of the oil are volatile.

The city water supply is drawn through a 66-inch brick tunnel extending out into the lake about 2400 feet. At times, the white water described above discolors the lake as far as the intake and the water is rarely without some odor or taste.

The problem for design was this, given a city water supply contaminated by a variety of unusual wastes, with oil waste predominating, to design a filter plant effective in removing odors and tastes as well as the bacterial content. The purpose of the testing station was to study this problem along the following lines: (1) The effect of aeration on the odor and taste. (2) The kind and amount of chemical to be used and its effect on the odor and taste. (3) The period of sedimentation and the time of contact giving the best results. (4) Any peculiarities that might develop in the treatment of water contaminated by oil.

The following summary of information gives data regarding the design:

Aeration. Size of collecting platform, 14 by 13 feet. Nozzle used, No. 9 Spraco. Pressure at base of nozzle, 8 pounds. Water rate, $8\frac{1}{2}$ gallons per minute.

Chemical treatment. Chemical used, alum, admitted between aerator and coagulation basin. Strength of stock solution, 10 per

cent. Strength of solution fed, 1.25 per cent. Method of feeding, through a small hole drilled in the side of a $\frac{3}{8}$-inch brass tube stuck through a rubber cork in the bottom of a wooden bucket. Head on orifice maintained by means of a fountain similar to the inverted water bottle often found in offices. Amount of chemical fed in water, from 0.6 grain to 1 grain per gallon of water treated.

Coagulation basins. Number used, 2. Size of each basin, 5 feet 6 inches x 8 feet x 2 feet 9 inches deep inside measurements. Depth of water, 31 inches. Capacity each basin, 825 gallons. Capacity in hours, 1 hour 40 minutes each when tanks were run in series (3 hours 20 minutes total) or 3 hours 30 minutes each when run in parallel. Number of baffles in each tank, 1 lengthwise at first with 2 passageways, later changed to have 3 baffles with 4 passageways. Velocity of flow, when run in parallel, 0.076 feet per minute with single baffle, and 0.172 feet per minute with triple baffle; when run in series, 0.152 feet per minute with single baffle, and 0.344 feet per minute with triple baffle.

Filter. Number, 1; area, 4 square feet; filtering rate, 8 gallons per minute; underdrain system used, Harrisburg type with pipe grid; depth of gravel, 8 inches; depth of sand, 30 inches; wash, hand-controlled from barrel storage; rate of wash, 15 gallons per square foot per minute; method of supplying wash water, hand pump lifting water from filtered water barrel to wash water barrels; method of rate control, float valve and orifice.

The results of aeration showed that the improvement in the water was not at all uniform. Its efficacy seemed dependent upon the direction and velocity of the wind which, when it blew across the plant of the Standard Oil Company, not only caused the air to become heavily laden with oily odors but also drove before it a great deal of sand and cinders which had become coated with oil. Under such conditions the spray actually washed oil out of the atmosphere instead of giving up that which it already contained. Nevertheless the aeration seemed helpful at all times for the water required a smaller amount of chemical to accomplish the final removal in the coagulation basins and filters, due perhaps to a breaking down of the emulsion.

Alum was the only chemical tried and it proved successful. As the oil which remained in the water after aeration was probably removed by adsorption, that is by sticking to the flocculent precipitate of alum, any other coagulent of the same general character

would doubtless have done as well. It was not thought worth while to go further into the matter. A fair average of the amount of alum required was about 0.7 grain per gallon of water treated.

In regard to sedimentation and the period of contact, it seemed that the oil required some time to attach itself to the coagulant, and when the floc was kept in suspension longer than in the common practice, a smaller amount of alum could do the same work.

Among the peculiarities which developed during the test, the most important was the necessity for frequent washing to maintain the normal rate of filtration. The filter acted as though it were air-bound. This was probably due to the oily floc forming a water-tight film over the surface of the sand, for it was discovered that by lightly raking the surface of the sand the filtering rate was restored and the loss of head returned to nearly normal. Later it was found that the same result was obtained by shaking the filter, i.e., by simply opening and closing the wash-water valve without actually washing the filter. In this way the period of service between washings was increased from six to sixteen hours, with a consequent reduction in the amount of wash water used.

Difficulty was also experienced in making satisfactory analyses of the samples. Since no other method was available for detecting oil in such minute quantities, appeal had to be made to the senses of taste and smell. Obviously, these senses are somewhat unreliable. However the tests were made independently by three different persons so that the results were well checked. It was found that it was impossible to judge odors in Whiting, because of the atmospheric conditions, so the samples were brought to Chicago for judging. Conclusions from the tests proved that the taste and odor could be thoroughly removed when the modifications of filter practice noted above were put into effect.

MIDLAND TESTING STATION

Another water testing station has just been constructed recently at Midland, Mich., by Pearse and Greeley.

Midland is a town of about 10,000 inhabitants near the center of a brine-well district. The Chippewa and the Tittabawassee Rivers unite in Midland. The Chippewa is less turbid and colored than the Tittabawassee, but has a drainage area more thickly settled and, consequently, much more polluted. Both waters are fairly

hard. At present, Midland obtains its supply from the Chippewa River, principally because of its clearness.

The problem at Midland is in part as follows: (1) Determination of the more suitable supply, the Chippewa or the Tittabawassee Rivers. (2) Determination of best method of reducing the color of the Tittabawassee River, if that supply is selected. (3) Determination of the best method for reducing the alkalinity of either source.

The testing station consists essentially of two coagulating basins, a filter, and a filtered water storage tank. Supplementary to these are an inlet orifice box, a divided orifice box, one portion of the flow passing through a lime saturator and joining the other portion, a mixing trough, wash-water barrels, and a filter-rate controller of the adjustable float type. The supply is obtained from the river through a small electric-driven centrifugal pump, and delivered to the inlet orifice box. The following summary of the different devices furnishes an outline of the essential features in the design and operation:

Lime. Admitted as lime water. Quantity used, approximately 8 grains per gallon. Method of feeding, by diverting a portion of flow (about one-tenth) through a lime saturator, and uniting it with the remaining portion as lime water.

Alum. Point of application, between the two coagulating basins. Method of feeding, a 20-liter bottle with side outlet controlled by adjustable screw cock. A glass tube through a cork in the top which terminated in an upturned gooseneck near the bottom supplied the air and regulated the head on the outlet. This bottle was capable of very fine adjustment.

Lime saturator. Size, 16 inches in diameter at the top, 4 inches in diameter at the bottom, depth 30 inches. Flow period, 30 minutes to one hour. Method of admitting lime, as a 10 per cent milk of lime mixed in barrel above.

Mixing troughs. Number of passages, 5. Each passage 3 inches wide, 3 inches water depth and 6 feet long. Period of mixing, approximately 3 minutes. Each passage was baffled with 1-inch cleats spaced 3 inches on centers and had a slope of one inch from end to end.

Coagulating basins. Number used, 2. Size and capacity, the same as of those used at Whiting. Period of flow used in tests, 8 hours and 6 hours.

Filter. Number of filters, 1. Material, galvanized iron. Filtering area, 18 inches in diameter, 1.75 square feet. Filtering rate, 3.5 gallons per minute. Underdrains, Harrisburg type.

Rate controller. Float-operated valve on effluent of filter.

Wash water. Supply, two barrels on a platform about 18 feet above the filter underdrains. Amount used in each wash, 100 gallons. Wash-water pump, electric-driven centrifugal.

The addition of 8 grains of lime per gallon with a sedimentation period of about 6 hours has demonstrated that the alkalinity of the raw water may be reduced from about 175 p.p.m. to 50 or 60 p.p.m. in the filtered water.

Unfortunately, on account of ice in the river, the tests on the reduction of color had to be discontinued but will be resumed at an early date. With lime alone the color was reduced from 45 to 15 p.p.m. Alum treatment has not yet started.

The saturator method of applying lime water works very well and, where such relatively small quantities of lime are required, it is much superior to the orifice box method of feeding a milk of lime solution. However, it requires a certain amount of care to determine when a new batch of lime shall be admitted to the saturator. The amount of lime wasted through inefficient saturation is negligible in a testing station.

A small laboratory was fitted up in conjuction with the station. Alkalinity tests on the raw water, lime water, coagulated water and filtered water were run at frequent intervals. Anything wrong or unusual in the operation was apparent to the operator at once. The solutions were made up so that no computations had to be made to interpret the results.

The testing station itself was housed in an old pump house, heated and lighted. This was a distinct advantage. The tests were run 24 hours of the day and thus approached actual working conditions.

COSTS

The accompanying table of costs of water testing stations is based on one published in "Sewage Disposal," by George W. Fuller, revised by Langdon Pearse in his lecture on "Experimental Engineering, Particularly the Construction of Testing Stations on Water and Sewage Problems" and brought up to date by the authors.

List of special American investigations on water and sewage purification

PLACE	DATE	WORK	COST
Lawrence, Mass	1887–11	Water and Sewage	$195,000
Providence, R. I	1893–94	Water	5,000
Louisville, Ky	1895–97	Water	47,395
Reading, Pa	1897	Water	1,500
Pittsburg, Pa	1897–98	Water	36,386
Cincinnati, O	1898–99	Water	41,588
West Superior, Wis	1898–99	Water	2,000
Washington, D. C	1899–1900	Water	8,000
Richmond, Va	1900	Water	2,000
New Orleans, La	1900–01	Water	23,606
Philadelphia, Pa	1900–05	Water	172,000
Springfield, Mass	1901–03	Water	18,000
Harrisburg, Pa	1903–04	Water	25,000
Columbus, O	1904–05	Water and Sewage	44,000
Oakland, Calif	1907–08	Water	19,390
Ontario, Province of	1909–15	Water and Sewage	61,900*
Detroit, Mich	1916	Water	8,000†
Cleveland, O	1916	Water	4,000‡
Milwaukee, Wis	1918	Water	40,000§
Whiting, Ind	1917	Water	1,000

* Includes about $14,000 for field work in Province.
† Does not include Operating cost of $9,000 per year.
‡ Does not include Operating cost.
§ Appropriation.

Construction cost of testing stations

STATION	SUPPLY	TOTAL	REMARKS
Water			
Baltimore	Gravity	$5,000	
Cincinnati	Gravity	19,717	Testing devices
		4,411	Laboratory building and equipment.
Detroit		8,000	
Harrisburg	Pumped	10,573	Including engine inspection
Jerome Park		7,825	(*)
Milwaukee	Pumped	8,282	Exclusive of piping, heating, etc.
Whiting, Ind	Pumped	500	
Midland	Pumped	500	
Cleveland	Pumped	4,000	

* Low bid, exclusive of filter washing machine.

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COST OF PUMPING THROUGH PIPE LINES[1]

By G. C. Habermeyer

Many municipalities secure water for a public supply some distance from the municipality. In a few places water flows from the source to the municipality by gravity, but in nearly all installations in this state it is necessary to pump through a pipe line. Water may be delivered directly to consumers or to a reservoir from which it is pumped or flows by gravity to consumers. The choice of a proper size for a pipe line is of considerable importance. Where reservoir capacity is not provided, pump and pipe line capacity should be sufficient to meet maximum demands. Occasionally high demand may be met in some installations by increasing pressure to correspond to increased friction losses in the pipe line, or, if water at low pressure will satisfy the extra demand, by allowing the pressure at points of consumption to drop.

In choosing a pipe line for any certain installation it is common to consider the economical velocity of flow through the pipe. This may be of considerable advantage, but it is probable that only on rare occasions will water flow at such a velocity. The loss of pressure in various sizes of pipes carrying water at a given rate of flow is often computed and the extra cost of pumping in the smaller pipes balanced against the extra first cost of the larger pipes. Officials in charge of the installation of, or changes in, a water supply would do well to consider the entire cost of pumping through the pipe line reduced to cents per thousand gallons or other unit by which water is sold. In some installations, knowing the comparatively low cost, more consideration would be given to secure a water of excellent quality at some distance, possibly from another public supply. In other instances the advantage of providing reservoir capacity to reduce fluctuations in the rate of flow or of providing good service with large-size pipe could be more readily shown.

In many cities one of the greatest difficulties in installing a public water supply is to secure active coöperation for the support of one

[1] Read before the Illinois Section, March 26, 1919.

definite project. Intelligent citizens in two small cities have been found to favor taking water from Lake Michigan more than 200 miles away. Figures on cost of pipe and pumping, estimating that with aid of other cities the water could be conveyed the first 150 miles at practically no cost, would have been rather easy to secure and would probably have convinced these people at once that another supply was preferable.

The life of a pipe line carrying water such as is in general use for public supplies is not generally affected by the velocity of flow within ordinary ranges. Therefore, to double the rate of flow divides by two the interest and depreciation charge per gallon. However, with increased rate of flow the friction increases; and, with same pipe line, the same cost of pumping 1,000,000 gallons 1 foot high, and not including interest and depreciation on the pipe line, increasing the flow 50 per cent multiplies the total cost of pumping by three.

A pipe line built without regard to increased demand in the future may result in very high pumping costs or building another line in the near future. A line built to give economical cost of pumping ten or twenty years in the future may be very expensive now on account of high interest charges on the investment. Since in many instances it is necessary to vary the rate of pumping throughout the day, the size of pipes should be so chosen that average cost will be economical. It may be economical in some cases to pump at a definite rate for several hours each day until sufficient water is pumped to meet the demand for the entire day. Interest and depreciation continue, however, when the line is not in use.

Knowing definitely the cost units, the cost may be shown best by curves, but for a basis from which to draw curves for any cost units, values are best shown by tables.

Table 1 has been prepared to show the cost of delivering water at uniform rates of flow through pipe lines. The pumping cost is taken at 10 cents per 1,000,000 gallons 1 foot high. Friction is computed by the formula of Flamant as given by diagram in "Public Water Supplies" by Turneaure and Russell. The cost of pipe is taken at $30 per ton for Class B cast-iron pipe and interest plus depreciation on the pipe line is taken at 6 per cent.

When pumping costs are low, pumping a little above the most economical rate does not greatly increase the total cost per 1,000,000 gallons. However, by choosing the pipe of next larger size the cost could, in many cases, be greatly decreased.

TABLE 1

Cost of delivering water through pipe lines by pumping at uniform rates

GALLONS A DAY	DIAMETER OF PIPE																																							
	6 inches			8 inches			10 inches			12 inches			14 inches			16 inches			20 inches			24 inches			30 inches																
	Operation	Six per cent of pipe-line cost	Total	Operation	Six per cent of pipe-line cost	Total	Operation	Six per cent of pipe-line cost	Total	Operation	Six per cent of pipe-line cost	Total	Operation	Six per cent of pipe-line cost	Total	Operation	Six per cent of pipe-line cost	Total	Operation	Six per cent of pipe-line cost	Total	Operation	Six per cent of pipe-line cost	Total	Operation	Six per cent of pipe-line cost	Total													
100,000	$0.42	$6.92	$7.34	$0.11	$9.12	$9.23	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....													
150,000	0.86	4.62	5.48	0.22	6.08	6.30	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....													
200,000	1.43	3.47	4.90	0.36	4.56	4.92	$0.13	$6.08	$6.21	$0.05	$7.81	$7.86	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....													
250,000	2.11	2.77	4.88	0.53	3.64	4.17	0.18	4.84	5.02	0.08	6.25	6.33	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....													
300,000	2.90	2.31	5.21	0.73	3.03	3.76	0.26	4.04	4.30	0.11	5.21	5.32	$0.05	$6.22	$6.27	....	....	....	....	....	....	....	....	....	....	....	....													
400,000	4.80	1.73	6.53	1.21	2.27	3.48	0.42	3.04	3.46	0.18	3.90	4.08	0.09	4.66	4.75	....	....	....	....	....	....	....	....	....	....	....	....													
500,000	7.10	1.39	8.49	1.78	1.82	3.60	0.62	2.42	2.84	0.27	3.12	3.39	0.13	3.72	3.85	$0.07	$4.50	$4.57	....	....	....	....	....	....	....	....	....													
600,000	9.77	1.15	10.92	2.46	1.52	3.98	0.86	2.02	2.88	0.36	2.60	2.96	0.18	3.11	3.29	0.09	3.75	3.84	....	....	....	....	....	....	....	....	....													
800,000	16.15	0.86	17.01	4.06	1.13	5.19	1.42	1.52	2.94	0.60	1.95	2.55	0.29	2.33	2.62	0.15	2.81	2.96	$0.05	$3.90	$3.95	....	....	....	....	....	....													
1,000,000	....	....	....	6.00	0.91	6.91	2.10	1.21	3.31	0.89	1.56	2.45	0.43	1.87	2.30	0.23	2.25	2.48	0.08	3.12	3.20	....	....	....	....	....	....													
1,200,000	....	....	....	8.26	0.76	9.02	2.89	1.01	3.90	1.23	1.30	2.53	0.59	1.55	2.14	0.31	1.88	2.19	0.11	2.60	2.71	$0.04	3.39	$3.43	....	....	....													
1,400,000	....	....	....	....	....	....	3.78	0.87	4.65	1.61	1.11	2.72	0.78	1.33	2.11	0.41	1.61	2.02	0.14	2.23	2.37	0.05	2.90	2.96	....	....	....													
1,600,000	....	....	....	....	....	....	4.77	0.76	5.53	2.04	0.97	3.01	0.98	1.16	2.14	0.52	1.40	1.92	0.18	1.95	2.13	0.08	2.54	2.62	....	....	....													
2,000,000	....	....	....	....	....	....	....	....	....	3.01	0.78	3.79	1.45	0.94	2.39	0.76	1.13	1.89	0.25	1.56	1.81	0.11	2.04	2.15	$0.04	$2.93	$2.97													
2,400,000	....	....	....	....	....	....	....	....	....	4.13	0.65	4.78	1.99	0.77	2.76	1.05	0.94	1.99	0.36	1.30	1.66	0.15	1.70	1.85	0.05	2.44	2.49													
2,800,000	....	....	....	....	....	....	....	....	....	....	....	....	2.60	0.66	3.26	1.37	0.80	2.17	0.48	1.11	1.59	0.20	1.45	1.65	0.07	2.09	2.16													
3,200,000	....	....	....	....	....	....	....	....	....	....	....	....	3.29	0.58	3.87	1.74	0.70	2.44	0.60	0.98	1.58	0.25	1.27	1.52	0.09	1.83	1.92													
3,600,000	....	....	....	....	....	....	....	....	....	....	....	....	4.03	0.52	4.55	2.13	0.63	2.76	0.74	0.87	1.61	0.31	1.13	1.44	0.11	1.63	1.74													
4,000,000	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	2.56	0.56	3.12	0.89	0.78	1.67	0.37	1.02	1.39	0.13	1.46	1.59													
5,000,000	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	3.80	0.45	4.25	1.32	0.62	1.94	0.55	0.82	1.37	0.19	1.17	1.36													
6,000,000	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	1.81	0.52	2.33	0.76	0.68	1.44	0.26	0.98	1.24													
8,000,000	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	3.00	0.39	3.39	1.25	0.51	1.76	0.43	0.73	1.16													
10,000,000	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	1.86	0.41	2.27	0.63	0.58	1.21													
12,000,000	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	2.56	0.34	2.90	0.87	0.49	1.36													
14,000,000	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	1.15	0.42	1.56													
16,000,000	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	1.45	0.37	1.82													

Costs for pumping part time at uniform rate may be computed from the table. For example, pumping through a 10-inch pipe line at a rate of 1,000,000 gallons a day, an average of eight hours a day, the cost of the pipe line per 1,000,000 gallons would be three times that given in the table or $3.63. Operation, if cost is 10 cents per 1,000,000 gallons 1 foot high, would be $2.10 as given; total, $5.63. Pumping for eight hours at a rate of 500,000 gallons a day and for sixteen hours at a rate of 1,000,000 gallons a day, the cost would be one-sixth of a million at $2.84 = $0.47, plus two-thirds of a million at $3.31 = $2.20; a total of 833,000 gallons at a cost of $2.67 or $3.20 per 1,000,000 gallons. For various costs of pumping corrections may readily be made. For cost of pipe delivered to the station within 1 or 2 miles of construction work other than $30 per ton, the cost may be closely approximated by varying the cost on the pipe line and the 6 per cent on investment proportionately, as the cost of iron is a large part of the cost and generally when pipe is high labor is also high. The actual cost of the pipe-line chosen for the table may readily be found, as 6 per cent of the cost is given.

DISCUSSION

M. L. ENGER: In considering only interest, depreciation and cost of operation, as these terms are usually understood, the author has omitted an important factor, the uncertainty of the future. This factor may be taken account of by an increase in the allowance for interest and depreciation as insurance against loss of principal and dividends. The amount of the insurance would depend upon the circumstances. Water companies will not make the extra investment required to put in a large pipe when it is shown by calculation, as given in the article, that a smaller pipe is as economical, unless satisfactory allowances have been made for possible future loss on the larger investment.

P. E. GREEN: That problem was one that had to be solved in the work of the U. S. Housing Corporation, particularly where the government had to deal with private water companies. Possibly there would be some 2 or 3 miles of large main to connect to the housing development. The corporation had G. T. Prince, formerly chief engineer of the Omaha and Denver water works, with it and he took up that problem. He reached the general conclusion that the most

economical size of pipe was one in which the velocity was over 4 feet per second. It seemed to work out very well.

L. R. HOWSON: Probably Mr. Prince's computation took into account that the work was temporary and that he could spend more on operation than he could in a plant which would be in service a long period of years. That was tried in a large number of cantonment jobs and it is the speaker's recollection that the figures showed for plants for permanent installation the economical velocity was nearer 3 than 5.

P. E. GREEN: In Mr. Prince's computations everything was intended for permanent construction. His figures were based on present prices but not on top prices. The best price was $50 to $55 for pipe. That is about twice what was paid in previous times, but although the high expense would tend to increase the most economical velocity, still all prices moved in parallel lines.

W. W. DEBERARD: It is possible that the coal price might go down but the investment in iron would not go down. The old figure of 3 feet per second is what was formerly used.

G. C. HABERMEYER: The economical velocity will probably vary a little with the size of the pipe. Only interest, depreciation and costs of pumping were considered in the paper, and it is well to keep in mind, as Mr. Enger suggests, that in an actual installation other factors must be considered.

NEW WELL OF LARGE CAPACITY AT THE UNIVERSITY OF ILLINOIS[1]

By M. L. Enger

The water supply of the University of Illinois is obtained from six wells in the glacial drift. The following table concerning the first five wells put in is taken from a paper presented at a previous meeting of this Section.[2]

	Well number				
	1	2	3	4	5
Depth, feet	138	140	142	140	140
Diameter, inches	8	8	8	12	12
Cost of motor pump and well	$934	$1359	$1487	$2106	$1701
Capacity, gallons per minute	60	69	81	71	75

Owing to the increased demand for water and because of trouble with the well pumps, the above equipment became inadequate. Accordingly, in October, 1915, the University entered into a contract with the Layne & Bowler Company, of Memphis, Tenn., for a well and motor-driven pump. The payment agreed upon was $19 per gallon per minute after one year's operation. It was stipulated that the minimum over-all efficiency of the motor and pump was to be 55 per cent.

The well was drilled by the rotary process, beginning January 17, and finishing May 5, 1916. The well is 36 inches in diameter to a depth of 160 feet and 30 inches in diameter for the next 9 feet. The driller furnished the following log:

	feet
50 feet of clay and gravel mixed	50
15 feet of clay and rock	65
30 feet of hard clay and sand mixed	95
47 feet of hard sand and gravel	142
26 feet of very hard and fine sand	168
5 feet of hard sand and a little clay	173

[1] Read before the Illinois Section, March 25, 1919.

[2] Some Costs of Maintenance of Motor Driven Well Pumps, Journal, June, 1917, p. 190.

The well was completed by putting in 98 feet of 24-inch, No. 8 gauge casing; 70 feet of 24-inch, No. 6 Layne patent shutter screen, No. 8 gauge; and 4 feet of shutter seal, 16 inches by 22 inches with a 6-inch cypress bottom. The space between the screen and casing and the wall of the well was filled with 1-inch gravel.

A turbine pump was placed in the well, and was started and stopped frequently in order to break down and wash away the mud wall which had been left by the drilling process, and also to carry away the finer particles of sand in the water-bearing stratum near the well. It was hoped in this way to form a cavity under the heavy clay roof, indicated in the above log, into which the gravel would flow and thus increase the capacity of the well. The driller states that about 30 cubic yards of gravel were fed in at the top.

A preliminary test showed a yield of more than 1000 gallons per minute with a draw-down of 53 feet, or a specific capacity of more than 19 gallons per minute per foot of draw-down. The well now has a specific capacity of 21.5 gallons per minute. Data concerning well 4 will be of interest in making comparisons. The well was tested immediately after its construction in 1906 and showed a yield of 108 gallons per minute with a draw-down of 15 feet, or a specific capacity of 7.2 gallons per minute. In 1916 the yield of this well was 71 gallons per minute with a draw-down of 14.6 feet, or a specific capacity of 4.9 gallons per minute. That is to say, well 4 in ten years of operation has had its specific capacity reduced more than 30 per cent, while the new well in about two years of normal use has had its specific capacity increased by about 10 per cent.

It was soon found that a yield of 1000 gallons per minute could not be maintained for any length of time without having the water level in the well and in the older wells gradually fall. A test was begun April 11, 1917, in which it was intended to run the pump continuously until the water level in the well should become constant. The water level went down quite rapidly during the first two days of the test and it continued to go down slowly until the morning of April 25, when some trouble at the switchboard caused the circuit breaker to blow out, stopping the pump. It was started again about twenty minutes later, but during the interval the hydraulic constants of the well had been changed greatly. Before the pump was stopped the discharge had been about 670 gallons per minute for several days. When the pump was started again the discharge was about 790 gallons per minute and twenty-four hours later it was about 710 gallons

per minute. The curves in figure 1 show the water level in the well and the discharge during the test. The change in the hydraulic constants was probably due to the fine sand drifting toward the well becoming entangled, gradually choking up the water-bearing stratum

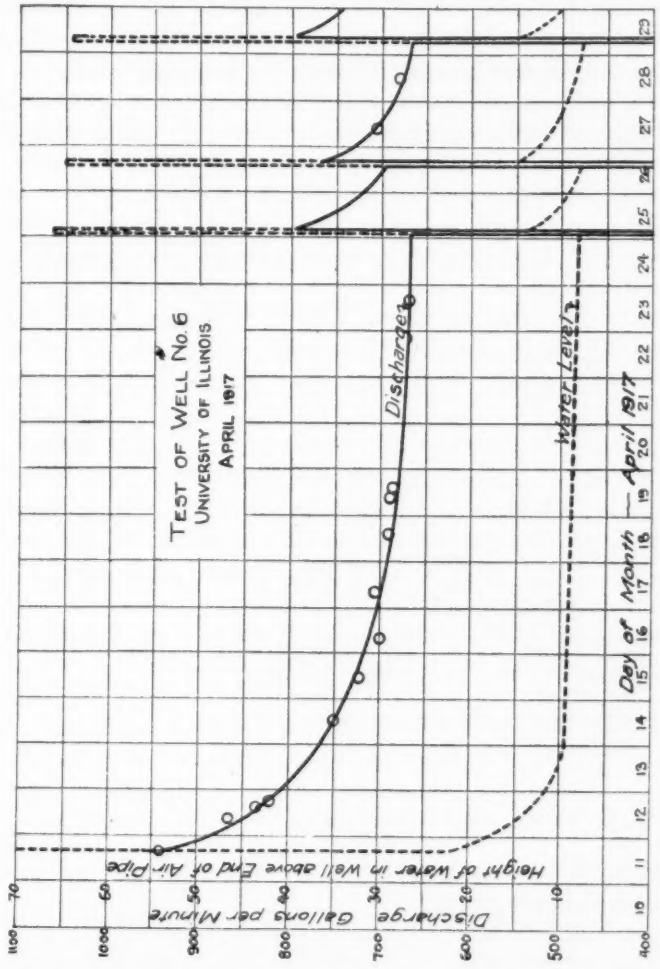
Test of Well No. 6
University of Illinois
April 1917
Discharge
Water Level
Day of Month — April 1917
Discharge Gallons per Minute
Height of water in well above End of Air Pipe

FIG. 1. DISCHARGE AND WATER LEVEL IN 24-INCH WELL DURING TEST

near the well. When the pump was stopped, the flow toward the well ceased, allowing some readjustment in the positions of the sand grains. When the pump was again put into operation some of the sand was washed into the well, leaving the water passages near the well less restricted.

The temporary installation was replaced by a six-stage, No. 4, 15-inch type CLC Layne pump driven by a 50-horse-power Westinghouse 2-phase induction motor through a flexible coupling.

The pump is located in a pit 12 by 12 feet, 11 feet deep, with concrete bottom, sides and roof. The pit is drained into a tunnel which carries the steam heating pipes into an adjacent building. Before the concrete floor was put in, the gravel between the casing and the wall of the well was replaced with clay to a depth of 6 feet below the floor of the pit. There seems to be no chance for contamination to

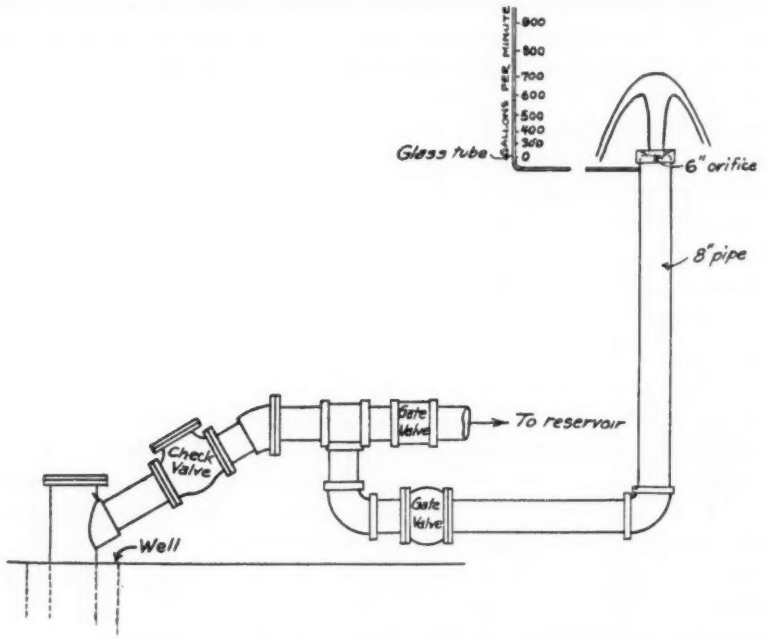

FIG. 2. APPARATUS FOR MEASURING DISCHARGE FROM WELL

get into the well. Frequent tests of the water have shown it to be of excellent sanitary quality.

In order to obtain continuous records of the water level in the well, a recording guage is used to record the pressure required to keep a small quantity of air bubbling from the end of a $1\frac{1}{2}$-inch pipe which hangs in the well and extends below water level at all times. The pressure in the air pipe is equal to the head of water above the bend of the pipe, hence the guage shows the submergence of the end of the air pipe at all times. The records show when the pump is started

and stopped and are found very useful in the preparation of operation statistics.

In order that the discharge may be measured at any time, a vertical jet apparatus has been installed in a manhole just outside of the concrete pit. The manhole is drained by a 12-inch vitrified pipe discharging into a storm sewer. The vertical jet apparatus was calibrated in the Hydraulics Laboratory by setting up the entire pipe line from the valve to the orifice. The equation of the orifice was found to be $Q = 530\sqrt{h}$ gallons per minute, h being the head in feet measured by a piezometer connected to the pipe 3 inches below the orifice. The pipe is 8 inches and the orifice is a 6-inch circular hole drilled in an 8-inch cap screwed to the end of the pipe. The general arrangement is shown in figure 2.

An acceptance test was made December 28, 1918, showing an average over-all efficiency of 55.6 per cent when delivering 587 gallons per minute against a head of 136.9 feet, and an efficiency of 59.3 per cent when delivering 536 gallons per minute against a head of 156.4 feet.

Compared with the University's other well equipment the new one has the disadvantage of a 15-foot greater draw-down. But since it has never been possible to maintain the average efficiency of the reciprocating well pumps as high as 50 per cent, the turbine pump will get the water out of the ground with a smaller power consumption than the reciprocating pumps. In the cost of maintenance the new unit seems to have a considerable advantage over the older ones. With the older equipment the cost of maintenance has amounted to about 0.8 cent per 1000 gallons. The new equipment has not been in use long enough to draw definite conclusions, but the indications are that the maintenance cost will be comparatively small.

DISCUSSION

C. B. BURDICK: There is no doubt of the practicability to increase greatly the yield per well in fine materials by the construction of a packed gravel well, such as has been described in the paper. Whether it is wise to construct such a well will always depend upon the cost and capacity of the well as compared with wells of small diameter and a lower cost in connection with the yield that is practicable to secure from them. Consideration should also be given to the cost

of pumping, considering the differences, if any, in the pumping machinery applicable to the wells of various diameters.

It is stated that the large well described in the paper cost about $10,000, and that its yield when first drilled was about 20 gallons per minute per foot of draw-down. It is further stated that the small tubular wells previously used at the University cost from $900 to $1500 per well and delivered, when pumped singly, about 7 gallons per minute per foot of draw-down.

Although the well described produced a large flow as compared to the wells previously used, its specific capacity is not great even as compared with tubular wells in the coarser underground strata. The wells supplying Camp Grant, constructed in 1917, were 10 inches in diameter, with the strainer from 10 to 20 feet in length, in sand averaging about 0.35 mm. in effective size. The specific capacities of these wells, pumped individually, ranged from 18 to 44.5 gallons per minute per foot of draw-down. The average delivery of five wells in a row 300 feet on centers, was 77 per cent of the sum of the specific capacities of the individual wells. The water-bearing stratum was not less than 30 to 50 feet in thickness. At Des Moines a 6-inch well in a 20-foot water-bearing stratum with sand 0.42 mm. in effective size, showed a specific capacity of 37. At South Bend, 6-inch wells in a 50-foot water-bearing sand stratum, in effective size ranging from 0.3 to 0.5 mm., showed capacities ranging from 31 to 38. At La Crosse, Wis., a 6-inch well in a 140-foot sand stratum, having a size of about 0.5 mm. near the strainer, showed a capacity of 23; a 10-inch well in similar materials a capacity of 37. At Benton Harbor, Mich., 6-inch wells in sand stratums from 20 to 60 feet thick, with sand sizes varying from 0.33 to 0.47 mm. in effective size, gave specific capacities from 22 to 44.

The coarser the water-bearing material the smaller is the gain by the construction of a so-called packed gravel well, and before constructing such wells it is proper to consider what can be done with wells more cheaply constructed, for the best well is the one that produces the desired quantity of water at the least cost per gallon, taking into consideration the initial expenditure and the cost of operation.

A. N. Talbot: The other wells of the University have a yield of 75 to 100 gallons per minute and it was thought that so high a yield as 500 gallons per minute could not be expected. The water-bearing sand is very fine, and it has given trouble in other wells.

P. E. GREEN: In the case of a deep well, in which the drop pipe is only slightly smaller than the outside casing, what is the best way to measure the draw-down? Suppose the space between the drop pipe and the casing is not large enough to put even a line pipe between them. One might get an insulated wire down, but the space is not large enough to get any pressure contact device down. The speaker thought that if he could drop a wire forming part of a bell and dry-circuit, the water would make a connection, but this did not work.

A.V. BEAUCHEMIN: A way to find the level of the water in the well is to take a small rubber tube and simply put the tube in the well between the casing and the discharge pipe. On the other end of the tube put a bit of glass U tube with water in it. When the rubber tube reaches the water in the well, air pressure will push water in the glass tube up. This is often better than electrical measurement. Sometimes a brass tube is placed at the end of the rubber tube to give weight to carry it down.

M. L. ENGER: On one of the University wells the space between the drop pipe and the casing is so small nothing can be put between them. After pumping, it is customary to wait until there is time for a sufficient leakage through the pump to bring the water level in the drop pipe or discharge pipe to the level of the water in the well, and then pump until water appears at the surface. If the slip of the pump and the capacity of the drop pipe are known and the number of strokes of the pump necessary to fill the drop pipe is determined, the depth of the water in the well may be computed.

C. B. BURDICK: Pumps in tubular wells should always be equipped with a water-level measuring device when installed, of which probably the best is a small pipe attached to the outside of the pump discharge pipe, from which the water level can be measured by means of compressed air and an ordinary pressure gauge, knowing the elevation of the lower end of the air pipe. Brass tubing $\frac{1}{8}$ inch in diameter, such as is used in gasoline lighting systems will answer this purpose. Where the well has not been so equipped the speaker has frequently used a telephone magneto and a weighted wire dropped into the space between the casings. The magneto is grounded on the pump casing, and will produce a ring where the water touches the water. It is necessary to protect the tip of the wire by placing it

inside a small piece of wall conduit. It can be held in place with paraffin and the tubing weighted with small shot so as to straighten out the wire. It is necessary, of course, to use weather-proof wire.

If the space between the casings is too small to admit the wire, it is practicable to use the annular space between the casings as an air pipe by closing the top of it and subjecting it to air pressure, then the location of the water level between the casings will be evidenced by the maximum pressure required to depress the water to the bottom of the pump. This reduced to feet of water and measured upward from the bottom of the pump will give the height to which the water stands outside the pump casing.

UNUSUAL GROUND WATER SUPPLY CONDITIONS AT BATON ROUGE, LOUISIANA[1]

By L. R. Howson

Many localities have no ground water supplies available at any cost, others but limited ones, and a few are blessed with adequate and satisfactory supplies from underground sources, but there is no section which can furnish ground water supplies more satisfactory in quality and under so many varying conditions of pressure, temperature, yield and depth as are found in the Mississippi River Delta near Baton Rouge, La. The Baton Rouge Water Works Company has been the pioneer in exploring the underground water resources of this locality, which now furnishes such an abundant supply for the city of Baton Rouge and the large oil refineries and other industries there.

The source of all ground water is rainfall, a large part of which is absorbed by the ground and travels slowly through the underground strata toward the water courses or other outlets. Based upon the gathering capacities of other watersheds where records are available, it is probable that from 20 to 40 per cent of the rainfall reaches the subsoil. Applying this minimum to the average Louisiana annual rainfall, each 1000 acres of porous strata exposed to rainfall collection will permit the withdrawal of 1,000,000 gallons per day continuously from the strata at some lower point without depleting the storage in the ground.

Louisiana and Southern Mississippi can be divided into three sections so far as the water collecting properties of the soil are concerned.

First. In the area from Baton Rouge south to the Gulf of Mexico the land lies at an elevation of 0 to 20 feet above the sea level. The area is level, quite marshy, and has a decidedly impervious topsoil. This area is of little value, either for ground-water collection or for its transmission, and accounts for the great depth to which New Orleans wells must go to strike water-bearing strata.

[1]Read before the Illinois Section on March 26, 1919.

Second. The area from Baton Rouge to a distance of 25 miles to the north rises from 20 feet to 100 feet above the sea level. The soil is sandy, the topography more rolling, and the area is excellent gathering ground for water. This area embraces about 1000 square miles.

Third. Just south of the Louisiana-Mississippi line the land becomes more rolling and rises quite rapidly to an elevation of from 200 to 400 feet above the sea level. The soil above elevation 100 contains a large proportion of gravelly material, and therefore presents a good exposure for the collection of rain water as the elevation increases. This surface condition prevails over nearly 20,000 square miles. The gravel strata are separated by tight impervious clay strata, which prevent the interconnection of the porous strata, and account for the variations in static pressure.

The porous soil which furnishes a good medium for the collection of the rainfall and its transmission extends over an area approximately half as large as the State of Illinois.

All of the gravel and clay strata dip toward the south at a slope of about 20 feet to the mile, so that the porous stratum outcropping near Jackson, Miss., at 400 feet above sea level is penetrated at Baton Rouge, 125 miles to the south, at 2050 feet below sea level.

With a knowledge of the above conditions, it would be natural to expect that plentiful ground water supplies would be available, and this is the case. At Baton Rouge, which is about 40 feet above the sea level, the Water Company's wells furnish the following data:

a. Wells 200 to 400 feet deep have a static elevation near sea level; the temperature of the water is 64°, and it is high in organic matter.

b. Wells 800 to 900 feet deep penetrate a 50-foot layer of sand, which yields a plentiful supply of water whose temperature is 72°. The static level is 35 feet above sea level, and tubular wells from 6 to 12 inches in size will show a specific capacity or yield for each foot the static water level is lowered, of from 10 to 12 gallons per minute.

c. Wells 1320 to 1350 feet deep penetrate a sand layer in which the static water level rises to 110 feet above the sea level. The water has a temperature of 85°, and wells have a specific capacity of but from 1 to 2 at the water works, and $6\frac{1}{2}$ at the city well recently drilled.

d. At 2050 feet a 60-foot sand layer is penetrated which produces wells having specific capacities of 12 to 15, and a temperature of 92°. The static water level is 160 feet above sea level, or 120 feet above the ground surface.

The Water Company had exhaustive tests made of its wells in 1916, at which time it was recommended that the future increase in supply be derived from the 2000-foot depth. Acting on that recommendation, the Water Company let a contract for drilling one well to that depth.

The well was drilled by the rotary process. It consists of 915 feet of 12-inch, 934 feet of 10-inch, 44 feet of 8-inch casing, and a 70-foot strainer. The latter consists of perforated well casing wound with brass gauze. The entire well was drilled and tested in sixty-two days, or at an average rate of nearly 34 feet per day.

No rock has been encountered in any of the wells sunk thus far, and the holes are cased the entire distance. In wells of this depth and high flow the friction becomes important, and it was thought at the time of the report that the largest casing that could be successfully handled would be 8 inches for the top 1300 feet, with 6 inches for the bottom 700 feet. It was estimated that such a well would yield about 840 gallons per minute with a free flow at the ground surface.

When the well was sunk, the contractor, who is one of the most experienced rotary well drillers, used the 12-inch, 10-inch and 8-inch sizes, and secured a flow of 1100 gallons per minute at the ground surface, the increase being due to the smaller friction losses in the larger casing.

The well has a static elevation 120 feet above the ground, or 160 feet above the sea level.

The contractor first ran a 10-inch fishtail drill down until he located a suitable foundation in tough blue clay for the 12-inch casing at a depth of 915 feet, then reamed the hole to 16-inch size, slushed the hole to insure its standing open, and inserted the 12-inch casing. This slushing is accomplished by pumping a mixture of clay and water to the bottom of the hole. As it passes upward outside of the pipe the clay particles seal the sides of the well hole and by their cohesive properties preserve the well bore until the casing can be inserted and lowered to position.

The assembling and lowering of 915 feet of 12-inch pipe required but $7\frac{1}{2}$ hours. The casing was allowed four days for the sand to set around it before drilling for the 10-inch casing was started, in a manner similar to that described for the 12-inch casing. Similar methods were followed for the 8-inch casing and the 70-foot strainer.

The drilling penetrated 14 separate sand strata, each from 5 to 194 feet in depth, all of which were separated from those adjacent

by thick layers of tough blue clay, which effectively cut off intercommunication of the water. Accordingly, in each layer the pressure of the water would depend upon the elevation of the outcrop of the stratum in which it was found.

In the report made to the Water Company on additional water supply it was recommended that the future supply be taken from the 2000-foot depth, and it was thought that the sinking of one or two wells would make it possible to shut down the pumps and supply the city of 25,000 people during the night by the pressure from the wells. One well only was sunk in 1916, and for about two years it has discharged directly into the distribution system for about 6 hours during the night, delivering at a 300,000-gallon rate and maintaining 30 pounds pressure. This practice has recently been discontinued on account of the pressure required for automatic sprinkler systems. If a second well was sunk, the two wells could furnish this amount of water with about 40 pounds pressure on the mains. During the day the well discharges into the suctions of the pumps which are under pressure up to a 1,500,000-gallon rate.

The ground waters of this district vary in temperature and pressure directly with the depth. The variation in temperature is important upon the well yields, for the fine sand of Louisiana with water at 92° will transmit the water nearly $1\frac{3}{4}$ times as rapidly as the same kind of sand in Illinois with the temperature about 50°.

All wells in Louisiana do not furnish good water, although those near Baton Rouge yield a soft water of excellent quality. At some points salt wells have been found. The occurrence of salt water is, however, confined to small areas and is probably accounted for by a depression filled with salt water and without an outlet left during one of the recessions of the ocean in the formation of the lower Mississippi Valley and the Gulf Coast. This underground reservoir is under pressure from its northern outcrop and when tapped yields salt water.

A large number of wells have been drilled in and near Baton Rouge in addition to that of the Water Company, the most important being the group of 14 wells drilled by the Standard Oil Company to supply cooling and condensing water for its oil refinery, which is now the largest in the world. These wells furnish nearly 5,000,000 gallons per day.

On account of the relatively low cost of sinking wells by the rotary process, a great many farmers have drilled wells to the 1300-foot

depth, and have water piped under pressure for farm and household uses. In pre-war times an 8-inch and 6-inch well 2000 feet deep, could be drilled, including casing, for $4000.

One peculiar phenomenon was observed in testing the wells in and around Baton Rouge. The tests were made at the time when the highest water stages ever recorded on the Mississippi at this point were prevailing. All of the wells tested in the 900-foot and 1300-foot strata had static elevations from 5 to 15 feet above those observed when the wells were drilled a year or a few years before, thus indicating that there is probably some connection between the river and the sand strata at some point between the outcrop and Baton Rouge.

It is probable that wells with still greater yield, higher temperature, and rising to still greater static elevations can be drilled near Baton Rouge, but, so far, 2000 feet is the greatest depth penetrated. The author knows of no other section of the country which has such a variety of ground water sources from which to choose its supply.

The author is indebted to W. P. Connell, President of the Baton Rouge Water Works Company, and to W. M. Eberhart, well contractor, for information concerning the methods and progress on the large 2000-foot well.

DISCUSSION

L. R. Howson: In reply to questions the author stated that any well water from the Baton Rouge district has a temperature which is too high for ordinary domestic use. The lowest temperature is 72°, from the 900-foot depth. The only other source than wells is the Mississippi River, and this was for many years the source used. The river was abandoned for the wells on account of the difficulty of operating the pressure filters. The summer temperature of the river water is also high. The question, therefore, is between using water of 72° or 92°, either of which requires artificial cooling before use for drinking purposes. The company pumps it into the mains at 92°. The water is very warm when drawn from the tap. There is very little temperature drop in the mains, which are laid at shallow depth.

The depth at which the temperature of a well begins to rise is largely a question of geological formation. Ordinarily, deep wells, even in this locality, have somewhat higher temperature than shallow ones. The unusually high temperatures at Baton Rouge, however, are due to the underlying deposits of sulphur and are typical of all wells in that locality.

THE WATER SUPPLY PROBLEM OF OUR STATE INSTITUTIONS[1]

By F. J. Postel

A brief outline of the administrative organization of the Illinois state government, as now constituted under the civil administrative code, will give a good idea of the nature of the work referred to the Department of Public Works and Buildings and the organization available for solving the problem. Attention is called to the similarity of the plan of organization to that of a large manufacturing industry.

The head of the Illinois state government is the governor. Under him are the nine department heads, known as directors, each in charge of one department. They are: director of finance, director of public works and buildings, director of public welfare, director of registration and education, director of agriculture, director of public health, director of labor, director of mines and minerals, and director of trade and commerce. Each of these departments is further sub-divided into divisions, the number depending on the number of sub-divisions most convenient for handling the business of the department. Thus, for example, the Department of Public Works and Buildings has seven divisions as follows: division of highways, division of waterways, division of printing, division of parks, division of purchases and supplies, division of supervising architect, and division of supervising engineer. The directors and heads of divisions are appointed by the governor. Their employes are all under civil service.

In a comparison with a large manufacturing industry, we then have the governor, comparable to the president and general manager; the directors, to the department heads of the business; and the division heads to the superintendents of the departments. Formerly, when the business of the state was split up among a large number of boards and commissions, each acting independently of the others, each could, and often did, do its own purchasing, hire its own

[1]Read before the Illinois Section on March 26, 1919.

attorneys, engineers, architects and other professional advisors. It is impossible to imagine a large manufacturing industry permitting each department head to have his own purchasing agent, his own lawyer, engineer and architect and to work independently of all the other departments.

If a department or division head needs supplies, a requisition is sent through the regular channels and lands in one of the four following places: (1) If it involves printing, it goes to the superintendent of printing. (2) If it involves engineering construction it goes to the supervising engineer. (3) If it involves architectural construction it goes to the supervising architect. (4) If it involves general supplies it goes to the division of purchases and supplies.

These divisions make the purchases, but they in turn may call upon any other division or department for assistance and advice.

For example, if the department of public welfare is in the market for blooded cattle for its farms, the division of purchases and supplies will call upon the division of animal industry of the department of agriculture for assistance in making the purchase. If the division of purchases and supplies needs data pertaining to coal, it calls upon the department of mines and minerals. If any department or division needs assistance in road construction, it calls upon the division of highways. If any department or division needs help in matters pertaining to heat, light, power, water, sewers, or similar matters, they call on the division of supervising engineer, which in turn may call upon the division of water survey or the department of health to work with it on the problem. The engineering division provides the means of supplying the water, but the division of water survey or the division of sanitation passes on the quality of the water. Briefly summarized then, the plan is to have each division specialize in some one branch of the state business, and then have all matters relating to that branch referred to that division.

With this outline of the organization, the manner in which this plan works out in the management of the state institutions may be explained as follows:

The welfare department has supervision over twelve hospitals for the insane and feeble minded, two penitentiaries, one reformatory, two correctional schools, three schools or homes for deaf and blind and three homes for soldiers and sailors, soldiers' orphans, and soldiers' widows. The department of registration and education operates five normal schools. The University of Illinois is not

included in this, as it does not come under the Civil Administrative Code.

In the operation of these institutions, the departments of public welfare and of registration and education occupy essentially the position of tenant, while the department of public works and buildings occupies the position of landlord. The landlord in this case, however, furnishes not only the buildings, but also all supplies, including food, clothing, equipment, heat, light and water. Under this plan of organization, it follows that the details pertaining to the supply of heat, light and water and similar service are under the supervising engineer. Under the plan of organization as it is now carried out, the design, construction, maintenance and operation of the mechanical equipment are treated strictly as engineering problems and worked out from an engineering standpoint, except, of course, for the one drawback which seems to exist everywhere, lack of funds.

Most institutions are located outside the corporate limits of cities so that, in general, they must provide their own source of water supply. We have in these institutions examples of practically every source of supply, including rivers, natural ponds, artificial ponds, deep wells and shallow wells and even springs. One institution, the Southern Illinois penitentiary at Chester, has three entirely separate sources of supply, the Mississippi River, an artificial lake in the hills back of the institution, and a large spring. The pumping equipment likewise represents practically every type used in plants of similar size.

When it is considered that the population of most of the institutions operated by the welfare department is from 1000 to 4000 and that these people live within the institution year in and year out, it will be realized that the problem of water supply for such an institution is very similar to that of a small town, except that the system covers a smaller area.

Unfortunately, few institutions are supplied with meters and in order to arrive at even an estimate of the water consumption per capita, it is necessary to use available data, some of which are perhaps not entirely reliable. Nevertheless, supplementing these with tests from time to time, it has been found that the water consumption per capita for the welfare institutions varies from about 70 to 350 gallons per capita per day. The 70-gallon institution was on a meter basis, while the institutions with a larger consumption,

as a rule, pumped their own supply. The author has been told at some of these institutions which pump their own supplies, that "our water costs us nothing as we pump it from our own wells." As a matter of fact, this statement is not quite so foolish as it sounds to an engineer for this reason. At most of the institutions, the principal load on the boiler plant is the heating load, and for about seven or eight months in the year there is no exhaust steam wasted. For all practical purposes then, the only cost of the water during the seven or eight months of heating season is the wear and tear on the machinery. At most of these institutions, much of the common labor around the power plant is inmate labor. However, the wasteful habits acquired during the seven or eight months of the heating season are continued during the remainder of the year and then it costs coal.

There is another aspect of the problem to consider. As the institution grows, its pumping capacity must be increased or its per capita consumption reduced. If reasonable economy is practiced in the use of water, the demand can often be reduced to a point where the same pumping equipment which supplied the institution in the past will be ample for the enlarged institution. If the waste of water were permitted to continue, it would entail the purchase of additional pumps in order to supply this waste.

One of the most interesting cases of water waste, and perhaps one of the most difficult to locate and stop, was at the Pontiac State Reformatory. All of the water for the power house, the laundry and the domestic hot water supply was pumped from a spring-fed lake on the institution grounds. Water for all other purposes was purchased from the local utility company. No records were available of the amount of water pumped from the institution lake, but the water furnished by the local utility company was metered. The author was surprised to find that this institution used 220 gallons per capita per day, exclusive of the water used in the power house, the laundry and the domestic hot water system. An inspection of the plumbing showed it to be in apparently good condition. The meter was tested and found accurate within ordinary commercial limits. The leakage from underground mains was so small as to be practically negligible, in spite of the fact that much of the underground piping was wrought iron and steel, installed for years.

The institution was finally divided into sections, so far as the arrangement of mains would allow. A meter was placed on each

section and read hourly for twenty-four hours as well as the meter on the service main. In this way it was found that the cell houses were the trouble makers. Each cell has a lavatory with a single self-closing faucet and a closet, flushed by means of a self-closing cock. One of the cell houses showed a per capita consumption of 140 gallons per day, while the other showed a per capita consumption of 72 gallons per day. Carrying the investigation further, it was found that the inmates had learned that by turning the self-closing faucet wide open, they could balance it in this position and by a little rough usage, could burr the top of the tooth or bevel, giving the pin a better surface to rest upon and thus making it all the easier to balance it in this position.

On arising in the morning, all the inmates apparently washed in running water, for during the first hour after rising, 434 inmates in the north cell house used 5500 gallons of water or 12.6 gallons per inmate per hour. In the south cell house, 543 inmates used 37 gallons or slightly less than 7 gallons per inmate per hour. The north cell house housed the more unruly boys while the other cell house was occupied by the better behaved boys. The all-night consumption ran rather high, indicating that a number of the boys were letting their faucets run wide open, presumably to have cold water should they awake in the night and want a cold drink.

A modified design for the self-closing cocks was worked out, which consists essentially of a stop so arranged that the faucet or cock cannot be opened into the wide-open position. The bar handle has been replaced with a round, smooth disc, having a knurled edge.

At the Watertown State Hospital, a pumping station is nearing completion which promises to solve the water problem there for some time to come. A year and a half ago, this institution had a new pumping station, built over a reservoir or basin, with a single small well opening into the bottom of the basin. It was quite complete, except that there was practically no water to pump. The small well was useful only as a test well, giving a clue as to the depth at which water was obtainable, the height to which the water would rise and the quality of the water. Having obtained these data, two wells were drilled, one on each side of the pump house. Well 1 is 275 feet deep, while in the case of well 2, it was necessary to go down to a depth of 325 feet to get the same flow and kind of water. In drilling these two wells, not over 70 feet apart, different strata were encountered all the way down. In the

case of well 2, practically the same water was obtained at a depth of 325 feet as in well 1 at a depth of 275 feet, only the flow was about 75 gallons per minute less. As the well was in solid rock at that point, the author decided to dynamite the well. He exploded 60 pounds of 40 per cent dynamite at the bottom of the well and succeeded in increasing the flow to just a trifle more than the flow of well 1.

The water situation at Jacksonville affects the engineering department directly because there are three institutions in that city, all depending on the city water supply. For years the water service there has been subject to interruptions and was extremely unreliable. A year ago, there was danger of a complete shut down of the supply and, in fact, arrangements were under way to temporarily close the institutions and transfer the inmates to other institutions, when the snow melted and relieved the situation. During that time, conditions had become so bad that it was necessary to shut the water off the buildings during certain hours in the day, as there was not enough to keep the plumbing in operation 24 hours per day. Snow was dumped into the institution reservoir and in some instances snow was brought into the buildings, melted in the bath tubs and the water used for mopping up the floors. During this time an insane hospital, a school for blind children and a school for deaf children were practically without fire protection.

As might be expected, the institutions which get their water supply from wells have considerable trouble with scale in their boilers and scale in their domestic hot water heaters. The problem of overcoming scale in boilers is an old one and, after all, resolves itself merely into a little engineering judgment and sufficient funds to carry out the idea. The problem of the domestic hot water supply is somewhat different for the department is limited in the kind of treatment it can give the water. Most of these institutions were equipped with domestic hot water heaters of the closed type, containing small brass or copper tubes. In several instances the tubes were practically stopped up with scale. To keep these heaters in service at all required frequent cleaning and even then, after a few days, the formation of scale in the tubes was sufficient to decrease the capacity of the heaters enough to cut down the temperature of the hot water perceptibly. Experience with this type of heater convinced the department that it was a failure with the water conditions existing at certain of the institutions.

Standard open feed-water heaters have been substituted for them because they are easier to clean and even when cleaning is neglected for a long period, the heating capacity remains practically unaffected. Instead of the small circulating pump ordinarily used with closed heaters, it is necessary to use a larger pump, one with sufficient capacity to supply the entire hot water demand. A centrifugal pump lends itself very nicely to this work, for if an unusually heavy demand is suddenly made on the system, the pressure falls and the capacity of the pump is increased proportionately. Moreover, the lower pressure results in a somewhat reduced flow at each outlet which, while not objectionable, results in the aggregate in an appreciable reduction of the demand. The characteristics of the pump, therefore, aid in increasing the capacity and reducing the demand simultaneously and automatically.

Where the open heater is used, the circulating line is connected back into the top of the heater. Circulation in the system is maintained by slightly opening the valve in the circulating line and discharging a continuous small stream into the heater. The only serious objection to this plan is the possibility of getting oil from the exhaust steam in the heater, into the water but by using a good grade of cylinder oil and a good type of oil separator and by properly designing the piping system, the probability of trouble from this source is small. This plan has worked out very successfully in every instance.

A discussion of the institutional water supply problem would not be complete without a reference to fire fighting equipment. This equipment is totally inadequate. Most of the institutions have an elevated tank for their daily supply. This is kept well filled and is the first line of defense. When a fire is reported, the fire whistle is blown, the fire pump started and the tank then shut off, giving direct fire pressure on the mains. In some instances, there are separate fire mains, but usually fire pressure has to be put on the entire system.

Every institution makes some attempt at an organized fire department and some of them are quite efficient. There was recently a midnight fire at an institution, in which, within five minutes after the first alarm was telephoned to the night operator, the fire pumps were in full operation and three lines of hose were playing on the fire. Except in one instance, a year ago when a water main laid too near the surface had frozen and was useless, the fire fighting equip-

ment and organization has handled every alarm promptly and effectively. Fire fighting is always serious business and in an institution which houses several thousand insane people, who are totally incapable of protecting themselves and are, moreover, often behind locked doors, the question of fire protection is doubly important. When you take away a person's liberty and lock him up, whether he is sane or insane, you owe it to him to protect him against fire with every means at your disposal.

DISCUSSION

F. J. POSTEL: In reply to questions the author stated that automatic sprinklers are not used anywhere, but their introduction in the broom factory at one of the institutions for the blind has been brought up for consideration. If such a system is justified anywhere, it is where blind people are employed in a broom factory. At the Watertown Hospital pumping station, the strata through which the wells passed varied all the way down. It was necessary to case one well only 25 feet and the other well about 70 feet. Furthermore, water was encountered at different levels. The static water level in the 275-foot well stands at practically the same elevation as in the 325-foot well. It has not yet been practicable to operate the two wells simultaneously. Permanent connections which will permit this are now being made. When well 2 was pumped the water level in the test well fell slightly and the water level in well 1 was practically unaffected, although it possibly did fall just a little. The water level falls as the rate of pumping increases. The limit to which the test went was the maximum drop in water level which would permit pumping with a suction pump having its cylinders located where the cylinders of the permanent pumping equipment are located, i.e., about 10 feet below ground level. The static level comes practically up to the pump cylinders or within about 10 feet of the ground level. The water level in the test well, which is located between the two wells, drops appreciably when either of the large wells is pumped and pumping one well does not materially lower the water in the other well 70 feet away.

The test well is a 4-inch hole to a depth of about 160 or 170 feet, and decreases in size at that point or else is obstructed at that point. The author was able, however, to get a sounding weight past the obstruction to a depth of about 260 feet.

D. R. GWINN: The speaker has in mind a city, which is not located in Illinois, where the fire protection equipment was out of order and could not be used. The condition was reported to the trustees, who, after some consideration, unanimously passed a resolution to the effect that the superintendent would be justified in having the necessary repairs made, in case there should be a fire.

PROBLEMS OF THE STATE DEPARTMENT OF PUBLIC HEALTH DURING THE WAR[1]

By H. P. Matte

The Illinois State Health Department has probably not been generally regarded as vitally connected with war activities. As a matter of fact, however, it has been instrumental in minimizing the evils of poor sanitation and the spread of contagious diseases, by cooperating with the United States Public Health Service in preparing an army of the highest physical and moral efficiency. In addition to contributing toward the efficient mobilization of troops, this department has raised the standards of living of both the rural and civic population. Improvement in public health administration stands out prominently as one of the results of the world war.

The establishment of national army camps near cities and small towns has forced these communities to recognize the necessity of protecting not only the health and lives of their citizens but also those of large numbers of soldiers and sailors. Military camps of some 40,000 or 50,000 men planted suddenly near towns of 15,000 to 50,000 population, together with thousands of workmen engaged in camp and in town work, as well as transient visitors, made the burden on the health organizations too great for efficient local handling.

The establishment of camps was not new business for the military authorities; but in the light of modern knowledge of sanitary science, it was recognized that it was not enough to safeguard the camp proper. The entire community which was directly or indirectly connected with the cantonments, must be included also. Of what use is it to protect the soldier from impure water in the camp, if the nearby town has a contaminated water supply which is furnished him when he is away on a pass?

The problem of camp sanitation, naturally was to be solved by the military sanitarians; but the case of the surrounding country belonged to the public. Early in the war the United States Public Health Service established extra cantonment zones which included

[1]Read before the Illinois Section on March 26, 1919.

all territory lying within a radius of 5 miles from the boundary line of the camp. It soon became evident that since the local authorities in small communities were not generally equipped to handle the situation effectively, some definite policy must be followed with the state departments and local boards coöperating.

1. The water supplies, both public and private including individual wells, must be investigated and improved when necessary.

2. Dairies inspected, the milk analyzed and its delivery supervised.

3. The sewage disposal facilities in town and in country districts investigated and made to conform to approved practices.

4. Garbage and manure disposal arranged for, as a protection against disease-bearing insects.

5. Stagnant ponds and marshy lowlands must be drained or otherwise treated to prevent the breeding of mosquitoes as a protection against malaria.

6. The housing facilities studied and the introduction of proper housing laws considered.

7. Surveys of schools made in connection with safeguards against all communicable or contagious diseases.

8. Investigations of all restaurants, lunch rooms, eating houses, barber shops, drug stores and places where soft drinks are sold, must be undertaken.

9. Review of all existing ordinances affecting public health and recommendations for new ones.

In Illinois, although the Federal authorities have sole jurisdiction on government territory, the outside district was cared for by the State Department of Public Health, coöperating with city and township bodies. In this state the government did not establish extra cantonment zones. This was probably because of the fact that it was early recognized here that there was need for intensive health work in connection with the mobilization of the army, because the work was started almost before the rest of the country had actually begun work on an organized basis.

The Illinois State Department of Health, coöperating with the various townships and boards of health, established what were known as military health zones. These districts were very flexible and not limited to the 5 mile radius. They included all cities, settlements, and country districts which were tributary and within reach of the soldiers. The zones comprized the vicinities of Camp Grant, Fort Sheridan, Great Lakes Naval Training Station, Chanute

Field, Scott Aviation Field, Rock Island Arsenal, and Camp Herrin, East Peoria, which was employed for instruction in the use of caterpillar tractors.

Each zone was in charge of a resident medical health officer from the State Health Department, who made his headquarters in the city nearest the military camp. The local, city and town health officers of the district were required to report to the resident representative, instead of to Springfield, in order to expedite service. An important work of these bodies was in connection with social diseases, which are generally recognized as most potent destroyers of army efficiency.

For this purpose, it was found necessary to establish a new division of the Department of Health, known as the division of social hygiene, acting under special rules.

The division of sanitation, the division of communicable diseases and the division of surveys and rural hygiene were called upon to carry on extensive health work in the sanitary zones connected with the government camps, while the division of tuberculosis supervised the medical care of soldiers who returned to the state infected with that disease. On account of the urgency created by the war, the peace-time activities of the Department of Health were seriously curtailed; and to make the work more difficult the Department suffered losses by the enlistment of the chiefs of divisions, engineers, district health officers and quarantine officers. Although some of the work originated in the Springfield department, the greater part was instigated by special and sudden demands on the part of the camp military officers and medical health officers in charge of the zones, who called freely upon all of the divisions.

Among the problems which confronted the State Department of Public Health and one which particularly related to water works operation, was the settlement of difficulties between certain cities and government boards. This work, being of a national character, did not always concur with local customs or ordinances, especially as regards quality and character of material and manner of installation. In many cases breaks between city officials and government representatives and contractors were imminent and the problems were referred to the division of sanitation. The local requirements and the alterations made necessary by war conditions were mutually adjusted. Appeals to the proper governmental authorities who were inclined to take a broad view when facts were understood, effected remedies.

Among special investigations was the sanitary survey of the Kishwaukee River, in order to determine whether or not soldiers could safely bathe in the river. The survey was an extended one, from which it was determined that the water would be reasonably safe by the time it reached Camp Grant, provided certain improvements were made in the operation of sewage treatment plants in certain towns above the camp.

Several investigations were made at Scott aviation field near Belleville with reference to mosquito control, the operation of the new sewage disposal plant at the camp, and the general sanitary conditions in the Scott field district of Belleville. This camp is located about seven miles east of Belleville. The natural drainage is poor, as about 1000 acres of nearly level ground comprise the site. Fifty-seven miles of farm drain tile had been laid and an existing water course was deepened from 2 to 10 feet for $2\frac{1}{2}$ miles, to provide a suitable outlet for the drains, with an available fall of 12 feet in the $2\frac{1}{2}$ miles.

Local farmers complained that the open ditch which received the treated effluent caused the flooding of certain lands and that the presence of sewage solids caused a serious nuisance. These farmers had been made to make privies fly-tight, to dispose properly of all garbage or other fly-breeding material, to screen rain-water barrels and cisterns or treat them to prevent breeding of mosquitoes, and to haul out manure from the stables and distribute it on fields so that no accumulation resulted. Many, accordingly, complained that inasmuch as they had been required to maintain their premises in a proper sanitary condition, the ditch from Scott field, with its decomposing sewage, should not be allowed to flow through their own fields. Although the seriousness of the situation was exaggerated, it was found that the sewage-disposal plant of the camp was overloaded. It was then altered somewhat in design, and much of the waste and storm water was diverted from the plant.

A sanitary inspection of East Peoria disclosed the fact that there was no provision in this town for sewage disposal. Upon the advice of the division of sanitation an ordinance was passed abolishing all privy vaults and requiring the use of the so-called dry-can system for the disposal of human excreta.

At Rantoul, much time was spent in studying the water supply and sewerage systems. Previous to 1917 neither odors nor nuisances were created by the discharges from the Rantoul outfall sewer. During 1918, however, odors were extremely foul and it was found

that the military camp was the cause of recent gross pollution. Remedies were suggested.

At Camp Grant, a study was made of the disposal of garbage and a general sanitary survey was made of the surrounding district by the division of surveys and rural hygiene, in coöperation with the division of sanitation, in order to prevent disease and establish sanitary reforms. An elaborate survey was made of Rockford, and several hundred farms outside the city were visited. This zone covered 150 square miles, four townships in one county and two in another. Particular attention was paid to food-handling establishments as well as to other phases of sanitation. The investigation comprised a follow-up system and actual improvements were inaugurated. This work included education in the proper disposal of human excreta and manure, prevention of fly-breeding places, construction of wells and the prevention of their pollution, construction of privies, care of barn yards, and protection against disease in the home and the school. All this work had a direct bearing upon the protection of soldiers against infection.

It was found that the average farmer does not appreciate the importance of the proper care of the manure pile, and of the many dangerous conditions found on the farms, the fly nuisance was the most prominent. Of eight hundred places visited 770 were found to maintain privies which were classified as insanitary. Privies were ordered abolished where connections with sewers could be had, and all premises along water mains were required to secure the benefit of municipal water supplies. Most of this work was duplicated with similar results in the other military zones. A general sanitary survey similar to that conducted in the vicinity of Rockford, was undertaken at Waukegan, North Chicago and the country surrounding Fort Sheridan and the Great Lakes Training Station.

It is interesting to note that the sickness rate in the military camps and zones in Illinois was very much lower than the average in the United States as a whole.

Through the excellent coöperation of officials and citizens, who soon readily appreciated the importance of the sanitary work and realized their responsibility, unprecedented reforms were inaugurated throughout the country. With the return of the soldier who has a new conception of right living and sanitation, these innovations will doubtless prove lasting and set a standard for the future United States.

QUALITY OF WATER IN ILLINOIS STREAMS[1]

By Minna E. Jewell

Heretofore the method of judging the sanitary quality of water of a stream has been to approximate the stream flow and the volume of sewage discharged into it. The errors of this method are obvious. In the first place, engineers do not claim any great degree of accuracy in such estimates as are ordinarily carried out. Second, sewages differ greatly in concentration. Third, granted the same concentration of sewage and the same degree of dilution, a small river purifies itself much more readily than a large one because of the relatively greater surface exposed to the air.

The United States Geological Survey published in 1910 mineral analyses of water from the principal Illinois rivers. The mineral analysis alone might tell us much concerning the quality of water in a natural unpolluted stream, but unfortunately there are few if any such streams in the state today, and the mineral analysis is no criterion to the quality of a polluted water.

Polluting materials may be roughly divided into two classes: (1) substances which harm the water by absorbing oxygen and producing toxic substances during their decomposition, such as domestic sewage and waste from starch plants, slaughter houses, and canneries, and (2) substances which are in themselves toxic, such as waste from gas works, mines and acid factories. Thus far the investigation of Illinois waters receiving waste of the second class (industrial waste) has been confined to studies of acid wastes from acid and munition plants, of which there are several in the state, and a few observations on mine waters. Due to the high alkalinity or hardness of most of the waters of the state the effects of acid wastes are purely local. The acid entering a stream is usually neutralized within a short distance of its point of entrance. It is pollution of the first of these two classes (domestic sewage and wastes which have a similar action) which is the problem of Illinois today.

[1]Read before the Illinois Section on March 26, 1919.

It is well known that as a stream becomes polluted the fish first to disappear are the more sensitive ones like trout, blue cat, eel and wall-eyed pike. As conditions grow worse these are soon followed by channel cat, mud cat, sucker, buffalo and native carp, and finally by such resistant forms as the dog fish, bull head, gizzard shad, and black bass, and last of all the German carp. But the presence of fish can not be used as a criterion of the damage done a river. At times of heavy rains fish may swim upstream or come in from tributaries, so that cat or buffalo may be found in a location where at low water even a German carp could not survive. It is known, however, that any water which will support cat or buffalo will also support the heavy shelled gill-breathing snail; bryzoa or moss animals, the same forms which sometimes cause so much trouble in water mains, should be abundant on stones, shells and submerged vegetation; fresh water sponges should be found on the underside of logs; while the delicate nymphs of the May fly, caddice fly and other insects should be abundant on stones in the riffles. These organisms can not migrate in or out of the locality, hence they tell the best story of conditions in the river. Whatever the quality of water at the time of the examination, unless an abundance of these organisms is found, we know that at some season the quality of the water is very poor, too poor for fish life.

The Sangamon River, upon which much of the author's work has been done, furnishes a typical example of a seriously polluted stream. It reaches Decatur as a small, relatively normal stream. Decatur derives its water supply from the river and as the demands of the town are greater than the flow of the stream at low water, there are several weeks each summer, and in case of a dry winter, a few weeks in the winter, when no water whatever passes the Decatur dam. This means that at such times the entire flow in the river below Decatur is crude undiluted sewage. As the Sangamon receives no important tributaries for at least 100 miles of its course, the changes which take place progressively are primarily due to the decomposition of the sewage.

Now what are the conditions in the Sangamon River shown by the biological survey? Not one of the organisms above mentioned is found between Decatur and Roby, fully 75 miles below. From Decatur to Harristown, a distance of about 20 miles, the only organisms found are moulds and bacteria; masses of slimy whitish-grey mould coat the stream bed and hang in heavy festoons from

every rock and twig, but not even sludge worms can tolerate the conditions which develop in this part during the late summer. Samples taken in only 3 feet of water last October showed no dissolved oxygen whatever.

At Niantic, some 15 miles farther down, the moulds are decreasing noticeably and the first sludge worms (tubificids) occur in the bottom. These worms are remarkable for their ability to live in the complete absence of free oxygen, just as some bacteria do. In late summer the only animals found free in the water are infusoria and foul-water rotifers, microscopic animals capable of living without ogygen and under conditions of rapid putrefaction, and minute round worms which are found in the floating tufts of mould. At the time of the spring rains, however, fish may be found at Niantic but these fish go down with the floods. Those which do not are soon gulping at the surface and dead along the shore. The odor of the Sangamon River between Harristown and Niantic can sometimes be detected at the distance of a mile.

From Niantic the number of tubificid worms in the bottom increases rapidly, so that in 10 or 15 miles the mud of the bottom seems alive with them.

South of Illiopolis, or about 50 miles by river below Decatur, the first trace of oxygen is found in the water of the bottom and with this one of the hardiest of insect larvae appears, the so-called blood worm or chironomid larva, frequently the cause of annoyance in city reservoirs. In a few more miles cray-fish may be found.

At Roby, about 75 miles below Decatur, there is a mill with a rather high dam across the river. This retards the movement of the water and allows the settling-out of organic matter to form a thick sludge on the bottom. Oxygen completely disappears from the deep pool above the dam, but as the water pours over the dam and rises in spray below, it becomes mechanically aerated so that shortly below the dam are found mussels or river clams, gill-breathing snails, fingernail shells and many kinds of insect larvae. In other words, the water below the dam is suitable for resistant fish such as German carp and bullhead. From this point, improvement is rapid. A short distance above Springfield, the relatively unpolluted South Fork enters, almost doubling the volume of water in the river. However, recovery is not complete even at Springfield. The more delicate organisms are still lacking. Below Springfield the river is again polluted by sewage, so it is not until Petersburg

is reached some 50 miles below Springfield that we again find normal conditions.

So much for the biological survey. The river is polluted, rankly polluted, the biological examination shows it, our eyes see it and our nose smells it, it is said, at a distance of two or three miles, but how can we measure quantitatively the amount of pollution at different points? At first, we might think dissolved oxygen would be a good criterion. Where there is no oxygen there are only sewage animals, and where oxygen increases the number of normal river animals increases, but that applies only to the later summer condition. Because of the higher solubility of gases in cold water and the slower decomposition of sewage, there is always an abundance of oxygen in the water during the winter except when the river is frozen over so as to exclude the air. Oxygen determinations made in February at Niantic, where the water was full of floating masses of bacterial slime and mould, showed 63 per cent of saturation, whereas only 54 per cent was found at Springfield in August.

Dissolved oxygen appears to be a fairly good index to pollution during the summer, and has the advantage of being readily determined in the field and requiring but little apparatus. The complete sanitary analysis, on the other hand, requires an equipped laboratory, but by the time a sample is collected and sent into the laboratory, say it requires a day or two in transit, it has undergone changes so extensive as to render it utterly worthless for this type of work. A sample of water allowed to stand in the laboratory six days showed over twice as much free ammonia as a similar sample analyzed immediately after collection.

Through the kindness of Mr. Spaulding, Commissioner of Public Works at Springfield, the author was able to set up apparatus in the meter-testing laboratory at Springfield, and could thus bring samples into the laboratory and begin the analysis before they had even changed appreciably in temperature. Of the various determinations of the sanitary analysis, oxygen consumed, nitrates, nitrites, etc., one in particular, the organic nitrogen, appears to vary according to the pollution. Starting with 0.7 p.p.m. at Decatur, we find a sudden rise to 2.2 p.p.m. at Niantic some 35 miles below Decatur, and then a gradual decrease to normal, 1.72 at Illiopolis, 1.2 at Roby, 0.8 at Springfield.

Samples of mud from the bottom of the river were also analyzed. It has already been remarked that the animals from the bottom are

a better index to the condition of the stream than the animals free swimming in the water because they are less influenced by temporary conditions, such as those caused by rain. The same thing is true of samples for chemical analysis. Mud samples taken after rains showed practically the same composition as those taken under normal conditions although the water samples were absolutely different. The ammonia and organic nitrogen of these samples were determined and (expressed as percentages of solid matter because there is always more water in some samples than in others) they show an abrupt rise below Decatur, followed by a gradual decrease to Petersburg, at which point the last sample was taken. Thus for ammonia we have Decatur, 0.0062; Niantic, 0.054; Illiopolis, 0.053; Roby, 0.021; Springfield, 0.0112; and Petersburg, 0.004; and for organic nitrogen, Decatur, 0.22; Niantic, 0.42; Roby, 0.27; Springfield, 0.224, and Petersburg, 0.11; so that if we plot concentration as ordinates with distance as abscissas we get a characteristic curve falling gradually at first, then very rapidly and then more gradually again to the normal.

The author is not prepared to say that any stream whose bottom mud shows over 0.01 per cent ammonia and 0.25 per cent organic nitrogen is seriously polluted nor that any stream whose mud shows over 0.02 per cent ammonia and 0.3 per cent organic nitrogen will kill off all but the hardiest fish and smell badly in summer. Work on a single river through a part of a year warrants no generalizations. Yet from the similarity of results already obtained in work on other rivers (the Embarrass, Muddy and Okaw are also under investigation), the author feels justified in the belief that by the time a number of rivers have been carefully studied for a period extending over a year some criterion will be found (perhaps not so simple as the example just given but still simple enough to be practical) for expressing quantitatively the pollution of a stream.

DISCUSSION

H. E. Babbitt: The author mentioned oxygen as purifying the water. May it not be possible that sedimentation is an important factor? Thus, the formation of the lakes on the Illinois River above Peoria gives a possibility for sedimentation and purifies the water.

M. E. JEWELL: The idea that running water purifies itself has a pretty firm hold on the popular mind. However, we find that the regions of a stream in which purification progresses most rapidly are where we have lakes alternating with riffles.

The statement that larger streams do not purify themselves as rapidly as smaller streams was based upon observations made upon the Illinois River last summer in the company of Mr. Richardson of the State Natural History Survey. The dilution of the Sangamon River is sometimes 0, sometimes 1 to 1 and sometimes 1 to 2. The dilution of the Illinois River is much greater, yet the Illinois River requires as long a distance to purify itself as the Sangamon does. If the Illinois River began as all crude sewage, as does the Sangamon below Decatur, the distance before purification would be accomplished would be much greater, probably not above Grafton. The reason the smaller river can purify itself more rapidly is because it allows more ready penetration of oxygen.

H. E. BABBITT: If the Sangamon River is worse than the Illinois, it must be terrible. The Illinois is as bad as possible below Joliet. The Illinois is a wonderful example of rapids and large lakes. The purification of the river is mainly due to oxidation until it reaches Peoria Lake, immediately above Peoria. This allows settling out and the water is in excellent condition when it reaches Peoria.

M. E. JEWELL: Coming down the Illinois there are no animals whatever in the bottom of the river, except sludge worms, until past Henry. From Henry to Lacon we begin to get a few so-called finger-nail shells. These animals are very resistant and can grow in the small amount of oxygen present. At Chillicothe we get our first appearance of gill-breathing snails. At Peoria narrows, we get the first appearance of the complete normal river bottom fauna. Sedimentation in the lakes is a most important factor, but an increase of oxygen also occurs there. Sedimentation is going on all up and down the river. The sewage settles on the bottom forming sludge, the decomposition of which is responsible for the depletion of oxygen at the bottom. The occurrence of oxygen at the bottom, then, means that the decomposition of this sediment is completed, at least to a certain stage.

At Hennepin last August, where there was no oxygen at the bottom, there was only a trace of oxygen at a depth of 6 feet.

although oxygen sufficient to support fish life was found at a depth of 1 foot. If water is 30 feet deep, the 1 foot of water at the surface which is supplied with oxygen will make no appreciable difference in the animal life. Bottom animals, as clams and snails, require less oxygen than the fish which swim in the water above, so allowance has already been made for the difference between the bottom and the water above. Where the bottom will not support these bottom forms, it has been found that the water will not support fish.

DESIGN OF A TILTING DAM AND ITS RELATION TO BACKWATER ON THE GUNPOWDER RIVER[1]

By V. Bernard Siems

The water supply of Baltimore is taken from the Gunpowder River, the headwaters of which lie in York County, Pa., near the Maryland line. The river then flows through Baltimore County, Md., in a southeasterly direction, emptying into the Chesapeake Bay about 10 miles northeast of Baltimore. The river, though small, has steep slopes, with a drainage area of 308 square miles above present dam. Its main tributary below the fall line is the Little Gunpowder Falls, which has the same general characteristics as the larger river. There are several small power developments along its course, of which the Warren and Phoenix Mills are the largest.

Previous to 1913 the City of Baltimore received part of its water supply from an impounding reservoir in the Gunpowder Valley, this supply having first been used in 1881. The works for taking this water consist of a stone dam about 25 feet high across the river, about 7 miles northeasterly from the city limits, creating a small reservoir, known as Loch Raven, with an original capacity of 510,000,000 gallons, and an area of water surface of 105 acres. The elevation of the crest of the dam is 171.2 feet. All elevations in this article are based on average mean tide in Baltimore harbor. At this time the city possessed the right to divert 170,000,000 gallons per day from the Gunpowder River. This capacity became largely reduced, by the deposit of silt, until in 1900 the reservoir had a capacity of 178,000,000 gallons. At this time dredging was begun, and continued varying in amount until 1909, with an average cost per year of $25,000. No record of yardage is available for the work was done by department forces.

Starting in 1906 the Water Department made plans for increased storage capacity.

[1] Revision by the author of a paper originally presented by title at the St. Louis convention, May, 1918.

Because of the fact that only $5,000,000 was available for the construction of a new dam and purification plant, the Department decided, in order not to affect the Warren Mills, that the elevation of the crest of dam was to be located at 192.0 (188.0 to 192.0 movable crest), giving a water surface area of 620 acres and a capacity of approximately 2,000,000,000 gallons, which is sufficient storage for an average daily consumption of 90,000,000 gallons, provided both the muddy and clear water were utilized. It was also decided to construct a purification plant to meet conditions for a dam at elevation of 240.0 and a pumping station to lift water to the filters, at an approximate cost of $5,000,000, including the purchase of flooded land (excepting Warren Mills), for a dam with its crest at 240.0.

This dam, built to an elevation of 188.0, is located about 2200 feet upstream from the old dam, and was designed for a width of base at the foundation line (approximately elevation 140.0 to elevation 153.0) adequate for a head at elevation 305.0 feet, this being the maximum height at which an impounding reservoir would be practicable. The area of water surface at this elevation would be approximately 9500 acres, giving a total capacity of reservoir of 142,300,000,000 gallons. From foundation elevation 155.0 to elevation 164.0 base for dam (spillway section) was designed for a head to elevation 240.0. The area of water surface at elevation 240.0 is approximately 2,391 acres, and corresponds to a total capacity of reservoir of 23,660,000,000 gallons. In 1911 the Warren Manufacturing Company filed an injunction prohibiting the building of this dam (crest 192.0). They foresaw a probability of injury to their water-power right at Warren Mills (tail race, elevation 196.0) should the City of Baltimore raise the proposed dam in any manner. The Warren Mills tail race is 7.92 miles above the proposed dam, measured along the axis of the river.

The investigations by the Baltimore City Water Department determined that the backwater curve for a discharge of 24,000 second feet over the crest at elevation 186.0 could not be considered as a governing point in the study of flood curves, because such floods would by themselves interfere with water power at the mill.

The backwater curve for a discharge of 5000 second feet over the crest of the dam (elevation 186.0 or water surface elevation of 192.0) would not show the water surface raised above the normal height at the Merryman Dam (crest elevation 194.3 and 6.80 miles above

proposed dam) of the Warren Mills more than it would be raised by the same quantity of water flowing with the Baltimore City dam not there. Five thousand second feet is over approximately 10 times the average flow and 3 times the maximum flow for the maximum month, and therefore is as large a flood as it is necessary to consider. Injury would not be done by a larger flood other than to affect the quantity of water power available during a few hours.

A flow of 930 second feet would not affect the Warren Mills with a solid dam to a crest elevation at 192.0.

The Baltimore City Water Department, after defeating the injunction, decided to build a masonry dam at elevation 188.0 and then an automatic superdam or tilting dam from 188.0 to 192.0. The masonry dam at elevation 188.0 was completed on November 1, 1913, and in 1917 the tilting dam was installed.

The present spillway has a length of 321 feet at elevation 188.0, and as this crest has keyways for future raising of dam, the center one was utilized for the location of the anchor block, which in turn acts as a pivot for the tilting dam.

To provide a tilting or otherwise movable super-dam in this spillway, in one unit, was impracticable, for structural reasons. More so, however, was this undesirable from the standpoint of waste of water. The tilting dam was arranged so that a portion would tilt when the water reached an elevation of 193.5, giving a depth of 1.5 feet of water on crest of movable dam. If the flood continued to rise, other portions tilt in turn, at different elevations.

The width of spillway was divided into 27 units or separate dams, numbered from 1 to 6. Each numbered section is arranged to tilt at a different height of water over crest. The summary below shows the number of each kind of unit, and the head under which they are designed to tilt:

NUMBER	UNITS	DESIGNED TO TILT AT
1	5	Elevation 194.0 feet
2	6	Elevation 193.5 feet
3	4	Elevation 194.0 feet
4	4	Elevation 194.5 feet
5	4	Elevation 195.0 feet
6	4	Elevation 195.5 feet

It should be noted that all sections are 4 feet (top of spillway to crest of movable dam) except section 1, which is 3 feet 9 inches high. The reason for making section 1 (composed of four 12 foot and one 9 foot sections, a total length of spillway of 57 feet, all at elevation 191.75) lower than the others was because of the desire to confine the normal water flow up to 192.0, also logs, floating ice, etc., to the center of the stream. Section 2 was designed to tilt under the minimum head of 1.5 feet because the intake channel, having an approximate elevation of 170 at the gate house, starts opposite this location (section 2), and therefore by an increased velocity due to the water being confined to this location, the deposit

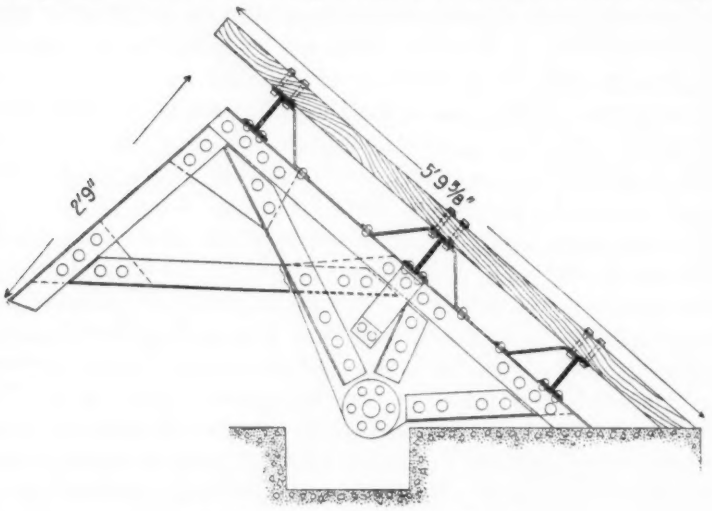

FIG. 1. STEEL FRAME OF TYPE 1

of silt at the end of the tilting dam will be kept away from this channel, and also the channel to the 48 inch sluice gate, whereas if section 1 was made to tilt at the minimum head the deposit of silt would fill the entrance to the channel. Note that section 1 and section 3 were designed to tilt at the same elevation.

The form of dam chosen was that of an obtuse triangle, pivoted at the obtuse angle, and forming a typical hollow or framed dam. As the water rises on a section its center of pressure on the inclined dam surface also rises, and eventually the pressure normal to the back will strike through the hinge. At this time, neglecting the weight of dam, the structure will be in neutral equilibrium. As the

pressure rises still further, the normal strikes above the hinge, and tilting takes place, the crest of the dam falling to take its second, or open, position.

The angle of the back was chosen as 40° with the horizontal, after several trials, as being the most economical of materials and the surest for operation. The horizontal center line of all the hinges is 2 inches above the original spillway, or at elevation 188.17.

The weight of frame 1 is approximately 1212 pounds. Water logging may increase the weight of dam to say 1400 pounds per section. The center of gravity of the dam in section 1 lies $8\frac{1}{2}$ inches upstream from the hinge and 1 foot $7\frac{1}{16}$ inches above the base, in this case, on the horizontal component of the pressure line when the dam is in a normal position. This means that the water must overcome a moment of $\frac{8\frac{1}{2}}{12}$ by 1212 pounds or 862 foot pounds before the dam will tilt, necessitating a further rise of water level sufficient to bring the point of application so far past the hinge that it will cause an unbalanced moment on the hinge.

It is significant that since these dams were installed they have been subjected to high water conditions, and sections 1, 2 and part of section 3 have tilted. Upon subsidence of the water these dams returned to their normal positions. In many cases the elevation varied from 0.3 foot to 0.6 foot lower than that for which they were designed. There can be numerous reasons for this, i.e., as the water subsides and reaches near elevation 188.0, foreign matter lodges at the toe, and when the dam drops back it seats on this obstruction, thereby throwing it forward. This is also the case when any warping of the timber decking at the toe and at the ends of the section takes place. In both cases the tendency is to decrease the head necessary for tilting. On the other hand, friction and corrosion at the pin offer resistance to tilting.

Note that as the dams tilt, the I-beams and timber decking offer some obstruction to the stream. This is especially true as the water subsides. The free waterway from the bottom of the lower I-beam to the concrete top of the dam (elevation 188.0) is 1 foot $2\frac{3}{4}$ inches, giving an opening of 1.2 by 321 feet or 385 square feet, obstructed only by the upright framing of the dam. The tilting dam usually drops back when the water reaches an elevation of 188.75 feet or slightly lower. This proves conclusively that the dams are entirely automatic in their action.

All sections are similar in detail, so that a description of one will cover all cases. Frame 6 (see illustrations) is composed of a deck supported on three I-beams, which are carried by two frames of angles and plates, 8 feet center to center, pivoted separately in cast iron bearings or anchor blocks.

The decking, 6 feet $2\frac{1}{4}$ inches long and 12 feet wide, is composed of first quality, $2\frac{1}{2}$ inch Georgia yellow pine. The I-beams are equally spaced on two frames, and bolted through their lower flanges, further stiffened against twisting by $\frac{3}{8}$ inch bent plates $6\frac{1}{2}$ inches wide, as shown, also bolted to the frame members. The frame is composed of 3 by 3 by $\frac{1}{4}$ inch angles and $\frac{5}{8}$ inch rivets. Pivot pins are 2 inch diameter tool steel turned to $1\frac{15}{16}$ inches. No

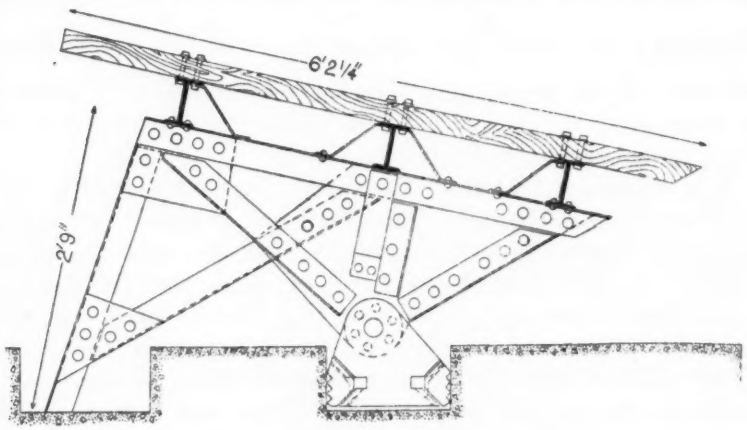

FIG. 2. STEEL FRAME OF TYPE 6

composition metal was used in the seats, though bearings of bronze would undoubtedly have been used had this work been carried on under usual conditions.

The castings used as bearings or anchor blocks for the dams have several unique features. The center keyway on top of the dam was utilized for anchoring the castings. It was necessary to cut the groove to 14 inches width at the bearings or anchor blocks, as the 12 inch original groove was irregular in alignment and did not give room for lining up the casting. The castings are different for each section of the frame. The distance from the back face of the keyway to the pivot for frame 1 is $4\frac{5}{16}$ inches, and for frame 6 is $8\frac{1}{2}$ inches. The groove for a width of 6 inches having been lined up on

the upstream side, the castings were all set with reference to this edge. The castings are three in number, a center block with a hole for the pin, and two side setting pieces and wedges. The center block was first set to position, then the side pieces slid in and tightened against the sides of groove by wedges. After aligning and wedging in position the spaces under the castings were filled with 1:2 grout, and the spaces between the side pieces and block were filled with lead. This gives an absolutely rigid construction, limited only by the strength of the concrete in the grooves of the present masonry dam.

The total cost of these tilting dams amounted to approximately $3700. This work was originally offered for contract, two bids being filed, one for $5275 and the other $9280. Since the estimate

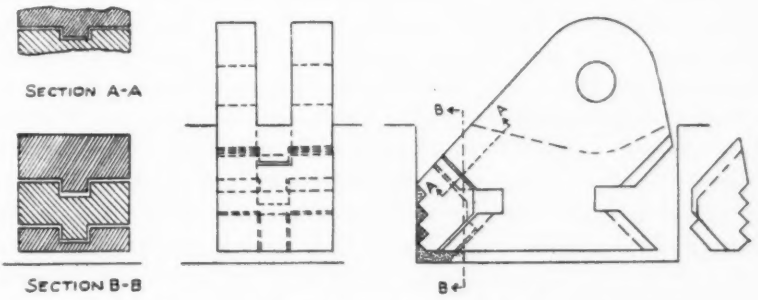

FIG. 3. ANCHOR BLOCK FOR TYPE 1 FRAME

made by the Water Department amounted to $3200, the bids were rejected and the work done by the Department's forces at the figure above mentioned.

By the construction of this tilting dam and the consequent raising of the water surface from elevation 188.0 to 192.0, a considerable saving in electric power consumption at the low lift pumping station was realized. With an assumed pumpage of 90,000,000 gallons per day the saving was approximately 1500 kilowatt-hours per day, or 547,520 kilowatt hours per year, and at a cost of 0.8 cent per kilowatt-hours, there is a saving of $4380 per year. These assumptions do not allow for the water surface lower than 188.0 or higher than 192.0, and, therefore, the author would say that the tilting dam paid for itself in not more than a year and a half.

The water impounded behind the new Loch Raven dam flows through a 10-foot circular steel conduit, paralleling the Loch Raven

Drive, until it intersects the 12-foot circular tunnel at the old Loch Raven gate house; thence through a 12-foot circular tunnel to the low lift pumping station at the Montebello filters, a distance of seven miles. It is of interest to note that although the 12-foot circular tunnel was built in 1881, when the amount of water consumed by the city was very small in proportion to the present consumption, it was designed and built of such liberal dimensions that it will be capable of supplying water to a population of at least 1,500,000, and is therefore a valuable asset to the city. The 12-foot circular tunnel is intercepted at the center of the pumping station by a concrete shaft 16 feet in diameter. This shaft extends vertically from the tunnel at elevation 145.94 to the floor of the pumping station at 206.25. The water level in shaft reaches a maximum elevation of 190.0 and varies, of course, according to the elevation of the reservoir at the new Loch Raven dam, the discharge through the tunnel and the loss of head. The shaft acts as a suction chamber for four centrifugal pumps, one 30,000,000-gallon, two 40,000,000-gallon and one 50,000,000-gallon per day capacity, which are arranged around it in radial lines. The water, after passing through these single-stage centrifugal pumps (center line, elevation 198.5) at a maximum rate of 160,000,000 gallons per day, is discharged into a 6.5-foot circular conduit, having a center line elevation at 201.5; thence through an 8-foot circular conduit to the head house, through a mixing basin, coagulating basin, filters and filtered water reservoir to the city distribution system.

When the dam at Loch Raven is raised to a crest elevation of 240.0 the centrifugal pumps will be by-passed and water from the shaft will pass directly through six 42 inch cast iron pipes arranged radially at center line elevation of 201.5 to a $6\frac{1}{2}$-foot circular conduit.

Due acknowledgment is made to John R. Freeman and Frederick P. Stearns, consulting engineers on the water-works improvements, for information received from their report to the Mayor and City Council of Baltimore in 1910; also to S. S. Field, city solicitor, for furnishing information regarding the case of Warren Manufacturing Company vs. The Mayor and City Council of Baltimore.

THE RELATION OF DEPRECIATION TO INVESTMENT[1]

By W. C. Hawley

This is a subject which until recently has received very little consideration from the management of most of our water companies. It was looked upon as theoretical, rather than a real thing seriously affecting the finances of the companies. Not many of our accountants and managers knew how to compute depreciation and very few water companies actually created and maintain a depreciation or replacement fund. We are only beginning, under public regulation, to realize the vital necessity of doing so.

Those of us who have owned automobiles for a few years have received practical and personal demonstration of the reality of depreciation. The machine that cost $2000 last year can be insured for only $1500 or less this year, and in from three to five years it must be replaced by a new one. It goes as part of the game; and when a new car is necessary we will write a check or mortgage the house and pay no more attention to it. But suppose by way of illustration, that a man in moderate circumstances invests his savings, and perhaps some money which he has to borrow, in an automobile and starts to operate a jitney or express route. Let us assume that he has no bad luck or accidents and that his earnings are sufficient to pay for gasoline, oil and repairs and give him and his family a living. At the end of five years we will assume that the jitney has to be scrapped. Then unless the owner has been setting aside from his earnings year by year a proper amount of money as a depreciation fund, he finds himself worse off than when he started. He has nothing to represent the savings which he put into the machine and the money he borrowed. His "plant" has been wiped out. He has no fund to replace it, and if he is to continue in business he must find new capital. This is only an illustration, but the principle is as true and operates as surely in

[1]Read before the Pennsylvania Water Works Association at Atlantic City on October 18, 1918, and printed in the Journal in order that the information may be available to the members of the American Water Works Association.

the case of the property of a water works company or any other utility as it does in the illustration given. It is not only recognized in our public service company laws, but, as we shall see, public service companies have been severely penalized in the past because they have not provided depreciation or replacement funds.

It is not necessary to consider carefully worded definitions of depreciation. Our chief difficulty is that the word has been used with so many different meanings. Depreciation as it affects us, is the reduction in value of our plants, due to age or change of conditions. There are various ways of estimating it, as the "straight line" method, "the sinking fund" method, the "equal annual payment" method, etc. When estimated for the period of one year it is spoken of as "annual depreciation," and when this depreciation is summed up for a period of years equal to the age of the unit considered, or the average age of the plant, we arrive at what is known as "accrued depreciation." Subtracting accrued depreciation from the original value of the plant, however that may have been computed, gives us the depreciated or present value of the physical plant.

What have we been doing in the past regarding depreciation? Many of us nothing, and most of us who have done anything have merely "charged off" depreciation. That is, year by year a certain amount has been added to the operating expenses and either the plant account reduced that much or a corresponding liability set up which was equivalent. Thus year by year the assets of our companies have been reduced and unless through excessive dividends an equivalent amount was returned to the stockholders, there has been no return to them of that portion of the original investment. In many cases the amount charged off as depreciation has been left in the business and practically it has been reinvested in the plant. The author will discuss this situation later.

The practical result of not creating a depreciation or replacement fund has been that when water works plants have been valued by the public service commissions, the historical cost has been determined where possible and also the cost of reproduction new, and from these and other considerations the value of the plant has been determined and from that figure the accrued depreciation has been subtracted. The Pennsylvania Public Service Commission, in the Beaver Valley Water Company case, subtracted an accrued depreciation of $215,175.11 from a "fair value new" of $1,187,286.11

which included, however, depreciable property to a value of $1,132,500.60. In the Ohio Valley Water Company case $88,661 accrued depreciation was subtracted from the reproduction cost of $1,083,825 (which included $10,000 working capital) while from the original cost of $841,343.00 accrued depreciation of $58,149 was subtracted. In the Westmoreland Water Company case from an original cost (including working capital of $10,000) of $1,266,-292.51 accrued depreciation amounting to $185,701.85 was subtracted. This was the amount of depreciation which had been charged off on the books of the company and which had been estimated upon a valuation much higher than allowed by the Commission, but it was not reduced accordingly. From a reproduction value of $1,373,904.54, estimated accrued depreciation of $122,-641 was subtracted. Thus we see the original investment reduced from 5 to 19 per cent; although not one dollar of this original investment has been returned to the stockholders. In the Westmoreland Water Company case, counsel for the Water Company attempted by argument to convince the public service commission that such a reduction was unjust. As a matter of fact, the money had not been taken out of the business, but had been invested in new plant to meet the public's demands for additional service. The practical condition was no different from what it would have been if the company had established a depreciation replacement fund by paying to a trustee or commissioner the amount charged off annually to depreciation and borrowing that money from the trustee, giving therefor the company's note and paying interest upon the money so borrowed. In a substantial water company this is probably one of the best methods of handling a depreciation fund. The answer of the Commission to this argument was as follows:

"The argument that the money represented by these charges remained in the plant for the benefit of the public service is not convincing. Some of it may have gone into the plant to offset depreciation, but the fact remains that in the past the company has not maintained a depreciation account, but has charged renewals to operating expenses rather than to such a fund. Practically all of these charges must have gone into extensions and betterments, and hence are included in the total original cost as found by the Commission. The money has been returned to the stockholders just as surely as though it had been paid to them in cash. They have built extensions and betterments with the money represented by these charges instead of with new capital and therefore, under the argument presented by the respondent, the deduction is entirely justifiable."

The flaw in this argument is so manifest that it needs little discussion. There had been little, if any, replacement necessary because of depreciation in the plant of the Westmoreland Water Company. Its original reservoirs, supply mains and distributing system were and are still in service. What few minor items had been charged to operating expenses rather than to the replacement fund, if any, could readily have been deducted and the accounts corrected. The Commission admits that "practically all of these charges must have gone into extensions and betterments." If such an expenditure can be considered a return to the stockholders, it follows that merely charging off depreciation is not a sufficient compliance with the principles of good accounting, or with what the Public Service Commission considers to be the proper procedure in such a case The decision of the Commission not only wipes out for rate making purposes the cost of additions to plant since it was originally built, to the amount of the accrued depreciation, but it leaves the Water Company without a depreciation or replacement fund. To demonstrate this statement let us put the facts into the shape of a formula, assuming that a depreciation fund had been created.

Let	P	= original cost of plant.
Let	A	= additions to plant paid for from charges to depreciation.
Let	a	= additions to plant paid for by capital other than from charges to depreciation.
Then	$(A + a)$	= total cost of additions to plant.
Let	D	= accrued depreciation.
Let	F	= depreciation or replacement fund.
Let	I	= total investment.

Then we will have

$$I = P + (A + a) - D + F$$

Since D and F are equal, this equation reduces to

$$I = P + (A + a)$$

In other words the original investment in plant and additions thereto has been maintained. Since no fund has been created, the Public

Service Commission says:

$$I = P + (A + a) - D$$

Assuming that the amount D has been completely expended in additions A
we have

$$A = D$$

and hence

$$I = P + a$$

This shows clearly that the company's investment has been reduced by the amount of depreciation charged off, and that the company has no asset to offset this loss. Surely this is not justice and does not accord with the decision of the United States Supreme Court in the Knoxville Water Company case which is quoted later.

Sound business principles require the setting aside of depreciation. The Pennsylvania Public Service Law provides:

> Article II, Section 1 (i). "It shall be the duty of every public service company to carry a proper and reasonable depreciation account, if required so to do by the order of the Commission."
>
> And also, (Article V, Section 15). "The Commission may, and shall after hearing had upon its own motion or upon complaint, establish by an order to be served, as hereinafter provided, upon every public service company affected thereby, a system of accounts to be used by such public service companies: and may also in its discretion prescribe the manner and form in which accounts, records and memoranda shall be kept by public service companies, including the accounts, records and memoranda of the conveyance of passengers and property and a proper and reasonable depreciation account, ..."

In carrying out these sections of the law, the Public Service Commission under date of March 25, 1918, prescribed and issued a Uniform Classification of Accounts for water companies, in which it provides for the keeping of an account to be known as "Depreciation of Structures and Equipment Reserve." This is on the asset side of the balance sheet, is a subtractive account and is identical with what has been generally carried as "Accrued Depreciation" on the liability side of the balance sheet. The Westmoreland Water Company was, therefore, carrying a depreciation account such as water companies are now required by the Public Service Commission to carry, and yet the assets of the company

were reduced by the amount of that account with no offset, and this was justified by the claim that the money had been returned to the stockholders because it had been invested in extensions and betterments to the plant. Of course, in obtaining the value of the physical plant it was perfectly proper to subtract the amount of accrued depreciation. The gradual reduction in the value of the plant because of depreciation is a fundamental fact, but if the original investment is to be maintained intact there must be a corresponding asset equal in amount, and this is the depreciation or replacement fund.

By way of illustration of what this may mean to the investors in a water-works enterprise, let us assume that the original cost of a plant is $1,000,000, and we will consider that no additions are made to the plant; also that it is a pumping plant of ordinary construction, and that to create a proper depreciation or replacement fund to be invested on a 4 per cent sinking fund basis, 1 per cent per year of the original cost of the plant must be set aside. This means an average life for the plant of about forty-one years. Under these assumptions the amount in the replacement fund at the end of each year would be as follows:

First year	$10,000
Second year	20,400
Third year	31,261
Fourth year	42,465
Fifth year	54,163
Sixth year	66,330
Seventh year	78,983
Eighth year	92,142
Ninth year	105,882
Tenth year	120,061
Fifteenth year	200,236

These figures are based upon the assumption that there had been no replacements made. This would probably be true for the first ten or twelve years, and we might then reasonably expect replacements requiring money to be taken from the replacement fund. If, however, at the end of fifteen years this company met with a rate case requiring the valuation of its plant, it will find its plant value reduced by 20 per cent of the original investment, and according to decisions up to the present, unless it has the equivalent funds in a depreciation or replacement fund it has

nothing to offset this loss. This is, of course, upon the assumption that the earnings of the plant have not been excessive and hence there has been no return of this amount or any part of it to the stockholders.

Against this method of valuation, we have the decision of the Supreme Court of the United States in the case of Knoxville vs. Knoxville Water Company:

"A water plant with all its additions, begins to depreciate in value from the moment of its use. Before coming to the question of profit at all, the company is entitled to earn a sufficient sum annually to provide not only for current repairs but for making good the depreciation and replacing the parts of the property when they come to the end of their life. The company is not bound to see its property gradually waste, without making provision out of its earnings for its replacement. It is entitled to see that from earnings the value of the property invested is kept unimpaired, so that at the end of any given term of years the original investment remains as it was at the beginning. It is not only the right of the company to make such a provision, but it is its duty to its bond and stockholders, and, in the case of a public service corporation at least, its plain duty to the public."

Surely nothing could be plainer than that, and yet in decision after decision of the various State Utility Commissions, accrued depreciation has been subtracted without any regard to whether or not it has ever been earned, or if earned it has ever been returned to the stockholders.

Arguments in the effort to convince Commissions of the correctness of this theory have been made so often, only to be ignored, that it seemed almost as hopeless as the effort to convince the Commissions that "going value," "cost of establishing business," or "the value of establisted business,"—whatever one chooses to call it,—has a separate and distinct cost and value over and above that of the physical plant. It was, therefore, with surprise and pleasure that the author noted the decision of the Wisconsin Railroad Commission handed down June 1, 1918, in the case of the Milwaukee Electric Railway and Light Company, et al., vs. City of Milwaukee. (P. U. R. 1918-E). In this decision the Commission says:

(11) "As reproduction cost new is not necessarily to be taken as representing fair value, it seems equally clear that reproduction cost new less depreciation cannot be so taken. What is to be ascertained is the amount fairly representing the prudent and honest investment made by the owners of the

utility. They are entitled to have that contribution kept intact until returned to them. In other words those that have contributed to the utility are, under normal conditions, entitled to a fair return on the amount of such investment while it remains in the utility. Whether that sum be arrived at on the original cost (physical property) or investment basis (and the two may be identical) the investor is entitled to a return on this amount, assuming that no part of his investment has been returned. If this amount, on the other hand, is taken as fairly represented by reproduction cost, the same principle would seem to apply; for reproduction cost less depreciation does not of itself establish the return of any part of the investment to the investor. Reproduction cost new less depreciation can be used as the final measure of fair value for rate making purposes only upon the assumption that the utility has not only earned the sum represented by depreciation in excess of a fair return on the original investment, but has also returned the same to the investors, retaining no depreciation reserve represented by assets. Where reproduction cost less depreciation is used as controlling in determining fair value unless the amount earned to provide for depreciation is returned to investors, it will not usually earn a return sufficient to constitute a fair return on the investment. Where the reserve is provided on a straight-line basis and retained by the utility, any interest earned on assets offsetting the reserve would constitute income of the utility. If the reserve assets are held in the business and the reproduction cost less depreciation is used as a basis for the valuation, a return upon such reproduction cost less depreciation at the normal rate, plus interest earned upon reserve assets, would be less than a fair return upon the investment unless a rate of interest were actually earned upon such reserve assets equal to the rate which constitutes a fair return for the property as a whole. Where reserve assets actually earn less than a fair rate of return upon the investment as a whole, the only way that occurs to us in which investors can obtain a fair return upon their full investment is by earning a higher rate on the cost new less depreciation than would be required if the investment, as already discussed, were used as a basis. Unless the rate of return on the cost new less depreciation is to be enough higher than the rate which should be applied to the investment to bring the total return, which is made up of the return on cost new less depreciation and interest earned on reserve assets, the reserve being provided on a straight-line basis, up to an amount which would constitute a fair rate of return on the investment, we see no way in which the investor, under such a method of valuation would secure a fair return on this investment, unless it is to be assumed that amounts earned for depreciation can be withdrawn from the business and the investment thereby diminished. This principle was fully established in the Knoxville Water Co. case."

Then follows the quotation from the Knoxville case which has been given above.

This is the clearest statement of the principles under consideration that we have seen, and gives evidence that not only are we progressing toward a full understanding of the principles involved

in valuation and rate regulation, but that our public service commissions when convinced of the correctness of a principle will deal justly with the public service corporations, even to the extent of reversing rulings laid down in former decisions. We may be dissatisfied with some of the things which the commissions have done, but let us remember that they and the utilities are working out a comparatively new proposition which promises much good both for the public and the utilities. We are traversing a path which has only been charted by broad statements of principles in the few legal decisions which have been handed down in such cases. Our courts in the few rate or valuation cases reported have avoided criticism by not giving the details of the calculations by which they arrived at values. Some such statement as "taking into consideration this and that and all of the other matters which bear upon this question, we find so and so" has protected many a valuation from an attack which should have resulted in reversal by the higher courts. The Commissions, however, as a rule have wisely adopted the plan of publishing the calculations by which they arrive at values. Some of the calculations are simple and the principles well understood and agreed upon. Others are involved, difficult of determination, with at least some of them still the subject of doubt and disagreement. Depreciation and its treatment in accounting and in valuation and rate making has been one of these. This Milwaukee decision is a milestone on the road of progress.

In the Uniform Classification of Accounts for water companies, prescribed by the Public Service Commission of Pennsylvania, to which previous reference has been made, we find that fixed capital has been divided into two accounts,—that installed prior to January 1, 1918, and that installed since December 31, 1917, the latter having been classified in accordance with prescribed accounts, and that the next account, No. 102, is called "Depreciation of Structures and Equipment Reserve." This, as previously explained, is a subtractive account, and in most of our bookkeeping has been called "Accrued Depreciation," and carried as a liability. This liability has been brought to the other side of the page and made a "red asset," by the same principle as that involved in algebra when we take a quantity from one side of the equation to the other, by changing its sign, and do not affect the correctness of the equation by doing so. This is something

new to most of us, but it is quite generally approved by expert accountants and is already in use by a large number of our great industrial corporations. The advantage of it is that by subtracting the account from the total of the fixed capital, there is shown at once the present or depreciated value of the physical plant. The use of the word reserve in this sense is probably unfortunate. The Century Dictionary defines reserve as: "That which is reserved or kept for other or future use; that which is retained from present use or disposal." The fund which is accumulated and set aside to pay for replacements is a reserve or reserve fund, but it is not clear how the accumulated losses in value due to depreciation can be called a reserve. However, the account known as "Depreciation of Structures and Equipment Reserve" is clearly explained in the printed text accompanying the Uniform Classification.

This explanation states that there is to be credited to this account and charged to account No. 402, "Depreciation," that amount which it is estimated will, through regular application, over the life of the structures and equipment of the various divisions of the utility water system, be sufficient to provide an adequate reserve to cover those expenses of depreciation that accrue upon such structures and equipment. This, and what follows, is clear and easily understood, but unfortunately neither here nor elsewhere do we find any statement regarding a depreciation or replacement fund, and yet, as we have seen, water companies have been practically penalized because they had not established such funds. It would seem as though there should have been provided a special place under the Asset Accounts for this fund. Inquiry of the Bureau of Accounts and Statistics was answered to the effect that the proper procedure in creating a fund would be to charge account No. 113, "Insurance and Other Reserve Fund Assets," subaccount —"Depreciation Reserve Fund," and to credit account No. 117, —"Cash." In other words, the amount charged to Depreciation each year is to be taken from Cash and put into a Depreciation Reserve Fund or a Depreciation Replacement Fund, which fund is to be carried as a subaccount under account No. 113.

William Morse Cole, in his book entitled "Accounts, Their Construction and Interpretation," gives (page 84) three sample Balance Sheets, showing methods of treating depreciation as follows:

BALANCE SHEETS

ASSETS		LIABILITIES	
First year—all plans			
Plant	$90,000	Stock / Bonds	$90,000
Second year—Plan I			
Plant	$80,000	Stock / Bonds	$90,000
Depreciation fund	$10,000		
Second year—Plan II			
Plant	$90,000	Stock / Bonds	$90,000
Miscellaneous assets increased	$10,000	Depreciation	$10,000
Second year—Plan III			
Plant	$80,000	Stock / Bonds	$90,000
Miscellaneous assets increased	$10,000		

Plan II he calls objectionable and Plan III seems even worse, and yet most of us have been using either one or the other of them. Commenting on them Cole says:

> To summarize what has been said of the depreciation fund, then, we find that it may appear on the balance sheet in one of three combinations: (1) among assets only,—in which case specific property is set aside to replace or repair machinery, or buildings, or what not, that are thought to have suffered actual depreciation; (2) among both assets and liabilities,—in which case specific property is set aside from net income as a safety fund for possible depreciation not thought to be actual; (3) among liabilities only,—in which case the amount is deducted from net income and shown on the books as a safety fund for replacement purposes, but the actual property is left among the general assets without specific designation. In the first case the fund could not be distributed to stockholders without impairing capital; in the other two cases the policy might be changed and the fund distributed without affecting capital, though such distribution might be morally and legally unjustifiable as a violation of implied faith.

Our Public Service Commission has adopted Plan I, and has improved on it by showing Plant or Fixed Capital at its original amount from which is to be subtracted the accrued depreciation, account No. 102, and the depreciation or replacement fund appears as a subaccount of account No. 113.

There is much of interest in Cole's book, which might be quoted here, but only the following paragraph will be added:

> Has it cost the corporation anything to accumulate this fund? If our real estate and plant are wearing out and are profitably employed, it is

obvious that they are reproducing themselves in the annual product. It follows, therefore, that, if the corporation cannot take from the annual product and lay aside as a depreciation fund the equivalent of the annual wear and tear of the real estate and plant, it is running down hill. In other words, that depreciation fund was created, day by day, in the regular product of the business. Real estate and plant, by constant use, have been slowly converting themselves from building and machinery into merchandise. It is the business of the accountant to see that that conversion is recognized and recorded. The surest way to keep it clearly in mind is to take the proceeds from the sale of some of that merchandise and set it aside as a special depreciation fund. Such a depreciation of real estate and plant is loss, of course; but it is loss only in the sense that consumption of raw material is loss; it reappears in the form of goods, and a certain part of the product must be recognized in that form as converted depreciation.

From the foregoing it seems evident that a water company in order to maintain its investment at 100 per cent has the right to earn year by year an amount sufficient to offset its loss by depreciation, and its duty to its stockholders and to the public to set this aside in a fund to take care of replacements as they become necessary. Whether the annual amount of depreciation should be based on the "straight line" or "sinking fund" plan is a matter about which opinion has differed. There are cases where the "straight line" plan should be used, but the consensus of opinion seems to be in favor of the "sinking fund" plan. The subject is discussed by the Wisconsin Railroad Commission in the Milwaukee Electric case with this conclusion:

"In our opinion a most equitable situation arises where depreciation is calculated and provided on a "sinking fund" basis. The fund should be in the custody of a commissioner or trustee, to be invested by him, subject to call for money with which to make replacements when necessary."

On this subject the Wisconsin Commission in the same decision says:

"The amount collected for depreciation can be treated in one of several ways. It can be loaned subject to call on very moderate rates of interest, being treated as a special trust fund represented by quick assets. It can be used from time to time to, temporarily at least, finance extensions and improvements. A considerable part of it, however, should at all times be immediately available for the purpose for which it is raised. However treated, it will be drawing some interest whether loaned or temporarily invested in extensions and improvements. Were it to be permanently invested in such extensions and improvements it would theoretically be earning interest

at the same rate as the original investment. As a matter of practice, however, this probably does not happen. As a rule, some time elapses after the expenditure before much additional return is earned on the new property. The fund is subject to be called upon at any time for the use for which it has been laid aside, and such extensions and improvements in a growing concern, at least, are usually financed, before the lapse of any extended period, by the issue of new securities."

In addition to these suggestions it would seem that in the case of a company which is strong financially, the trustee could lend the funds in his hands to the company on its note or notes, at the low rate of interest at which the annual payments have been calculated, reserving a sufficient amount on hand, on deposit or loaned on call, to take care of such ordinary replacements as are likely to occur. If some unusual demand for replacement should be made, the trustee could demand payment by the company of a sufficient amount on its note or notes to meet the demand for cash from the replacement fund. This would seldom occur, and if it did, the company would, in the meanwhile have had the advantage of the use of funds at a low rate of interest instead of having to sell bonds or stock or borrow from bank at bank rates of interest. The time when new capital had to be procured would thus have been deferred.

There is one other fact in the consideration of this subject which we must grasp clearly and insist upon. That is that the investor has the right to a reasonable return not only upon the depreciated value of the plant, but upon the replacement fund; for this fund is what maintains his investment at its original amount or value. But, it will be argued: "The fund is earning interest and you are asking a double return on it." The answer to this argument is that because the fund earns interest the annual amount paid into the depreciation fund is less than it would be otherwise, and the interest that the fund earns becomes part of the fund and is not returned to the investor. Both the annual payment and the interest are necessary to create a fund sufficient to make replacements as they are required, assuming that the calculations on which the fund is based are correct. This would not be true if the fund were created on the "straight line" basis, as was pointed out by the Wisconsin Commission in the Milwaukee case. Assume that a plant has cost $100,000. There can be no question of the right of the investors in that plant to earn a reasonable return upon that amount the first year. Suppose that the depreciation amounts to

$10,000 in the first year, and a replacement fund of that amount is set aside from earnings. Shall the reasonable return for the second year be based on an investment of $90,000 and this amount reduced year by year? If the balance sheet shows,

ASSETS			LIABILITIES	
Account number				
100, 101	Fixed Capital	$100,000		
102	Depreciation of Structures and Equipment Reserve (Red)	10,000		
	Depreciated Plant Value	90,000	Stock, Bonds	$100,000
113	Depreciation Replacement Funds	10,000		
		$100,000		

the answer is plain.

Since preparing the foregoing, the author's attention has been called to the decision of the Indiana Public Service Commission, Re: United Public Service Company of Rochester, Ind., P. U. R., 1918-F, page 316. In passing upon the matter of depreciation the Commission comments as follows:

"The Commission is confronted with the fact that, after five years' operation of the law in Indiana, the intent of the depreciation provisions has not been realized. The practice, both in this state and elsewhere, is to dissipate these moneys.

In estimating allowances to be made to cover depreciation, the Public Service Commission of Indiana adopts a rate allowance which presupposes that moneys provided to meet deferred depreciation shall not lie idle. Such moneys create the "depreciation fund." If this fund compounds itself, then the rate of allowance to cover depreciation may be made lower than otherwise. In other words, a fund so administered means a lower rate to the patrons or consumer than if such fund is not interest bearing. For the utility, the creating of such an actual depreciation fund constitutes an asset that should favorably influence its credit. In times of stress or disaster a utility having such a fund may be able to either borrow from this fund or convert or pledge securities held in such fund, and thus be relieved of heavy discounts otherwise incurred.

Sound and desirable as is the theory, the practice in this and other states is almost universally different. Depreciation moneys are paid out in dividends or used in other ways. The result is that in may cases the "depreciation fund" is a meaningless bookkeeping account. There is no reserve

fund, though the patrons have provided money for such a fund. The public utility is, instead of being financially fortified, reduced to the practice of merely charging off annually on its books a certain amount of the value of its property, reducing rather than maintaining its property value. Often this is finally reflected in poor service. The consumer who has provided proper funds is penalized in service because of improper administration.

The appended order provides that this utility shall take out of its revenues—the same as it will take moneys for operation, taxes, interest, and dividends—the money allowed and collected under the law, for depreciation; that it will pay such moneys into a depreciation fund; that such funds shall be held intact and separate at all times; that there shall be paid out of such funds all cost—including labor as well as materials—of replacing depreciation; that the funds shall be administered with proper accounting; that interest on such depreciation funds shall accrue not to the utility, but to the fund; that petitioner itself may, with limitations, borrow from the fund, but in such event shall place in said fund its own obligations, in the form of notes or bonds bearing interest payable to the fund.

The order does not contemplate that, when depreciation funds are borrowed by the utility itself, there shall be a mere bookkeeping account. In other words, the moneys derived from allowances to cover depreciation that are made in rates are to be regarded as a trust fund provided, as prescribed by law, by those served for the benefit of the utility, but not in reality belonging to it until such moneys are translated into actual plant replacements; and such fund must be handled, at the utility's risk, to the best advantage of the consumers and patrons and stockholders and bondholders.

INFORMATION FROM PUBLIC SERVICE COMMISSION DECISIONS RELATIVE TO FIRE PROTECTION[1]

By J. N. Chester

The accompanying tabulation of fire protection charges made against municipalities by different public utility commissions will, no doubt, prove interesting to many managers of private water companies, especially those who have in the past wondered whether or not they are receiving from the municipality the full amount to which they are entitled, and to others who realize they must have an increase in rates. To them it may serve as a partial guide to that portion of the burden that should be allocated to fire service.

It will be noticed that the charges are mainly from Wisconsin, there being but one from Indiana and one from Illinois, but it is interesting to see that these two states have followed close in the footsteps of Wisconsin.

It would probably be better, however, to those contemplating activities in rate regulation, to more thoroughly study Pennsylvania decisions, and to assist them the author has prepared the tabulation on page 322 of five different decisions that have been rendered by the Pennsylvania Commission, setting out and featuring only those items incident to the fire protection charges.

The author's firm has for its own use prepared from these tabulations curves from which it can forecast beyond the limits. With more time to tabulate additional decisions, it is hoped to make these curves of great value and assistance in the work of dividing the gross burden between the municipality and the domestic, commercial, and industrial consumers.

[1] Read before the Pennsylvania Water Works Association on October 18, 1918, and printed in the Journal of the American Water Works Association in order that it may be available for the use of the latter Association's members.

Fire protection charges made against the municipality by Public Utility Commissions

DATE OF DECISION	CITY	STATE	POPULATION, 1910	PER CENT OF WATER WORKS CHARGED TO FIRE SERVICE	VALUE OF PLANT	GROSS REVENUE	REVENUE FROM FIRE	PER CENT OF GROSS REVENUE FROM FIRE
1913	Elkhart Lake	Wis.	950	59.0	$8,425		$ 600	
1912	Sharon	Wis.	1,100	*52.0			1,000	
1914	Waterloo	Wis.	1,106	60.0	30,500		1,500	
1913	Fennimore	Wis.	1,275	*55.0	23,046		1,500	
1914	Elroy	Wis.	1,450	47.4	32,215		1,500	
1915	Barron	Wis.	1,674		35,000		1,392	
1912	Evansville	Wis.	1,963	64.1	37,122	$4,813	2,500	52.0
1914	Vaughn	Wis.		53.0	38,000	6,860	2,800	40.8
1917	River Falls	Wis.	2,300	54.6	56,000	*9,429	*3,240	34.4
1913	Delevan	Wis.	2,450	64.0	42,291	6,700	2,800	41.8
1913	Columbus	Wis.	2,523	53.2	53,696	6,499	2,600	40.0
1910	Jefferson	Wis.	2,582	75.0	51,524	6,208	2,560	41.2
1915	Prairie du Chien	Wis.	3,179		78,400	5,418	2,800	51.7
1913	Fort Atkinson	Wis.	3,300	51.0	73,424	9,156	3,600	39.3
1910	Ripon	Wis.	3,811	65.2	103,101	12,923	6,082	47.1
1913	Sparta	Wis.	3,973	55.0	98,599	11,047	4,400	39.8
1911	Oconto	Wis.	5,629	57.5	125,000	19,031	9,800	51.5
1912	Neenah	Wis.	5,734	54.5	117,038	12,850	5,850	45.5
1916	Merrill	Wis.	8,689	47.0	200,000	*30,415	11,000	36.2
1914	Watertown	Wis.	8,829	47.2	202,677	23,394	8,500	36.3
1915	Brazil	Ind.	9,340	70.0	128,383	18,987	6,361	33.0
1917	Lincoln	Ill.	10,892	43.0	215,000	29,850	9,651	32.4
1914	Ashland	Wis.	11,594	54.5	480,000	65,000	21,000	32.3
1911	Janesville	Wis.	13,894	54.0	250,000	38,724	15,800	40.8
1911	Marinette	Wis.	14,610	38.5	326,759	47,630	14,800	31.1
1911	Beloit	Wis.	15,125	48.0	289,512	41,565	14,687	35.3
1915	Beloit	Wis.	15,125	48.0	402,000	60,323	16,659	27.6
1915	Appleton	Wis.	16,773	45.0	525,000	50,616	19,000	37.5
1912	Eau Claire	Wis.	18,310	48.5	314,987	*46,655	15,000	32.2
1913	Green Bay	Wis.	25,236	48.7	683,229	*88,665	34,000	39.8
1910	Madison	Wis.	25,531	49.6	650,000	62,260	20,000	32.1
1914	Sheboygan	Wis.	26,398	46.0	507,739	69,176	27,500	39.8
1912	Superior	Wis.	40,384	46.0	838,242	110,146	27,900	25.3

* Estimated.

Decisions of Public Service Commission of Pennsylvania regarding fire protection

YEAR OF REPORT	NAME OF COMPANY	POPULATION SERVED	VALUE OF PLANT	GROSS REVENUE	PER CENT OF GROSS FROM FIRE	PER HYDRANT	PER MILE	REVENUE FROM FIRE	EQUIVALENT PER HYDRANT
1916	Beaver Valley	40,000	$985,000	$135,950	18.5+	$7	$400	$25,000	$64.24
1917	Portage	4,000	132,520	*8,935		15	175		
1918	Portage		Increased to		16.2	15	195	1,453+	51.91
1917	Ohio Valley	45,300	924,744	135,138	20.0	7	329	27,034	67.92
1917	West'md	60,955	1,100,000	156,520	17.3−			26,500	
1917	West'md Increased		1,125,000	158,270	17.1−	9	485	26,500	97.07
1918	Springfield Consol.	Approx. 150,000	8,251,420	899,732	18.9−	7	355	169,893	137.45+
	Springfield Consol.	Including Private Fire			20.0+			180,130	

* $8,935, within the borough and $16,113 total. The decision referred only to revenue from fire protection within the city but the Commission found that the total revenue within the borough should amount to $12,881, to give a fair return.

WILL HIGH PRICES CONTINUE?[1]

By Irving Fisher

At the present time there is a marked halt in production. Industry is slowing down. Unemployment of labor increases. Some industrial concerns are failing to earn profits, and others are suffering the dissipation of their accrued profits, because, even by shutting their plants down, they can not save certain of their expenses or any of their fixed charges The Government's revenues, dependent as they are upon the national income, may fall short at the very time we need them most. In brief, we are threatened with a widespread business depression and from peculiar causes, for the unsound conditions usually preceding a widespread business depression are absent.

The main reason why business is not going ahead better is that most people expect prices to drop. The merchant is selling, but not buying. The manufacturer holds up the purchase of his raw materials. People quote the disparity between present prices and those prevailing "before the war," and decide they will not buy much until present prices get down to "normal." This general conviction that prices are sure to drop is putting a brake upon the entire machinery of production and distribution. Readjustment waits because we keep on waiting for it. We have waited in vain for over three months. It is interesting to observe that

[1] During March and April many conferences of men prominent in commercial and industrial circles were held with heads of various departments in Washington, for the purpose of accelerating the resumption of peace-time activities. The chief cause of delay in starting construction work on a large scale had previously been ascribed to high prices for materials and labor. Irving Fisher, professor of political economy at Yale University, who had been advising the Treasury Department on monetary matters during the war, was accordingly invited to prepare for the use of these conferences, an explanation of the reasons given by economists for resuming construction now, without waiting for a fall in prices. His discussion is printed here by the Publication Committee, in accordance with the desire of the federal departments concerned in commercial and industrial affairs, to bring the subject to the attention of all public officials in charge of construction of any kind.

many manufacturers think that prices must come down, including the price of labor; but they are ready to demonstrate to you that their own prices can not come down, nor can they pay lower wages. Almost everything they buy somehow costs twice as much as before the war, and their labor is twice as dear. They can not pay their labor less if labor is to meet the increased cost of living. Now, as a matter of fact, when we investigate almost any individual one of the so-called high prices for industrial products we are likely to find that individually it is not high; that is, it is not high relatively to the rest. Our quarrel is with the general level of prices.

Variations in the general price level may be compared to the tides of the sea, while individual prices may be compared to waves. Individual prices may vary from this general level of prices for specific reasons peculiar to individual industries, just as the height and depth of waves vary from the general level established by the tide. The causes controlling the general price level are as distinct from those controlling individual prices as the causes controlling the tides are distinct from those controlling individual waves.

All prices have risen, but some have risen more, some less, than the average for particular reasons affecting each industry. In some cases an improved organization of both employers and employees has enabled them to combine against the public and take full advantage of the price advance. The war brought about an abnormal demand for certain products like copper and steel, and they advanced faster than the average. The abnormal demand having disappeared, these prices are being adjusted downward. Wheat is a case where demand increased and at the same time certain of the usual sources of supply—Russia, Australia, and Argentina—disappeared, with a resultant abnormal price increase. The closed sources of supply have opened again and wheat prices in the world market have dropped. In some cases, as in many of the industries making building materials, the war meant a great slackening in demand, an enforced curtailment in use by Government order. In such instances we are likely to see an upward swing in prices as the suppressed demand again makes itself felt. To-day we are witnessing throughout the country such price readjustments, up and down, but the general price level has shown little sign of falling, as is evidenced by price index numbers. It is apparent to every thoughtful observer that some great force has affected all prices, creating a new standard to which they are all conforming.

The fundamental practical question confronting business men is whether the general level of prices is going to fall. In my opinion, it is not going to fall much, if at all. We are on a *permanently* higher price level, and the sooner the business men of the country take this view and adjust themselves to it the sooner will they save themselves and the nation from the misfortune which will come if if we persist in our present false hope.

The general level of prices is dependent upon the volume and rapidity of turnover of the circulating medium in relation to the business to be transacted thereby. If the number of dollars circulated by cash and by check doubles while the number of goods and services exchanged thereby remains constant, prices will about double.

The great price changes in history have come about in just this manner. The "price revolution" of the sixteenth century came upon Europe as a result of the great influx of gold and silver from the mines of the New World. Europe was flooded with new money. More counters were used than before in effecting exchanges and prices became "high." People talked then of *temporary* "inflation," just as they talk of it now. But it was not temporary; it was a new price level.

A similar increase in prices all over the world occurred between 1896 and 1914, following the discovery of the rich gold fields of South Africa, Cripple Creek, and Alaska, the invention of the cyanide process in mining, and the vast extension of the use of bank credit.

Circulating credit—that is, bank deposits subject to check and bank notes—is a multiple of the banking reserve behind these deposits and notes; and the essence of this reserve is gold. Our present monetary system is an inverted pyramid, gold being the small base and bank notes and deposits being the large superstructure. The superstructure grows even faster than the base. The deposits are the important elements. They are transferred by check from one individual to another; that is, the circulation of checks is really the circulation of deposits.

Thus any increase in the country's gold supply has a multiplied effect. The possible extent of that effect is dependent upon (1) the amount of gold available, and (2) the gold reserve requirements, determining the volume of credit that can be put into circulation based upon the gold. Over a billion dollars in gold has come into

this country from abroad since 1914, and a large amount has disappeared from domestic circulation. The gold from both these sources has found its way into the United States Treasury and into bank reserves. On June 30, 1918, the portion of the *gold* reserve of the Federal reserve banking system which supported national bank deposits and Federal reserve notes was more than three times as large as the gold reserves under the old national banking system on June 30, 1914—$1,786,000,000, compared to $592,000,000. During the same period credit instruments (demand deposits and notes) increased about twofold—from $6,100,000,000 to $11,700,000,000. This increase of credit instruments is typical of the banking situation for the country as a whole and largely explains the present high level of prices. The increase of gold has been so great, however, that the base has grown faster than the superstructure—which is contrary to the normal tendency. The ratio of gold to credit has risen from 9.6 per cent to 15.3 per cent. The legal reserve requirements of the present system are such that for 1918 there is an excess of gold above these requirements of more than $700,000,000. The reserve required by law to support the $11,700,000,000 of credit instruments of 1918 is $1,070,000,000. The $700,000,000 of free gold could support an additional superstructure 70 per cent as large as the existing one, which indicates that for the banking of the country as a whole a potential future expansion of 50 per cent is a conservative estimate.

Many people, referring to this inflation in the circulating medium, and assuming that it is temporary, are waiting for this inflation to subside. When we speak of *inflation* we mean more circulating medium than is needed to transact the business of the country on a given price level. But what price level? Some people mean the price level of 1913–14. Our currency is certainly inflated in terms of the prices of that period, just as the currency of 1914 was inflated with respect to the prices of 1896, but our currency is not inflated at the present time relative to the new level of prices in the world which the war has brought. The country's volume of money will have to be judged in terms of this new price level, not in terms of a price level that is past. To speak of the present "inflation" as temporary is to assume the very thing about which we are contending—to assume that the normal prices are those of 1914.

Let us examine the factors upon which any future price movements must depend.

1. Gold will not return to circulation. No great effect in the direction of falling prices can be expected from any return of gold and other lawful money into daily circulation. Such a reversion would be contrary to monetary experience everywhere. When people have learned to leave their gold and silver in the banks and use paper money and checks instead they find the additional convenience so great that they will never fully return to the old practice.

2. No great outflow of gold through international trade. It should be noted that many of the former reasons for a flow of gold from America abroad have disappeared. We used to owe Europe a huge balance of interest payments upon American securities she held. The situation is reversed to-day. Moreover, Europe must pay us money for the materials we will send her for reconstruction, or at least pay us interest on credit we will extend her. Thus our exports will probably exceed our imports during the reconstruction period. We used to pay ocean freight money to foreign carriers; to-day the American merchant marine will keep in American hands tens of millions of dollars of ocean freight money. The huge volume of American tourist travel abroad, for whose expense we had to settle, has stopped and can not resume for a year at least. For all these reasons the lines are laid for a movement of gold from Europe here rather than a movement of gold from America to Europe.

"Yes, but," people say, "wait until trade is resumed between the United States and Europe, then surely 'low-priced European goods' will flow over here in such enormous volume that they will liquidate all annual obligations to us in goods." Ultimately Europe must pay her obligations to us in goods, but it will take many years. Meanwhile she needs our tools, machinery, and raw materials for immediate reconstruction.

At the present time European goods are not "low priced" (however little the money wages of European labor will buy). Prices in Europe since the war began have risen more than they have in the United States. The price rise has been less the farther from the seat of hostilities. It was least in Australia and New Zealand. It was next least in the United States, Canada, and Japan. Then came neutral Europe; then our present allies; and finally Germany and Russia. Gold tends usually to flow from high-priced countries to low-priced countries, so that until "inflated" European

prices fall gold is not likely to flow thither. Prices are no more likely to fall there than here, and for the same reasons which will be explained below.

3. Reduction of outstanding credit. The chief dependence of those who predict lower prices is on a reduction of the superstructure of credit resting upon our gold rather than on any reduction in the volume of this gold itself. They look for a contraction of bank credit, a reduction in the volume of deposits subject to check, which circulate throughout the country.

But the main cause for the present extension of bank credit is the liberty loan, and there is soon to be another. Subscribers for the new loan will not pay for their bonds in full any more than they did in the previous cases but rather less. Many of them will deposit the bonds with the banks as security for loans to be repaid later. The effect on our circulating medium will be the same as if the Government were to impose a levy of $6,000,000,000 of credits upon the Federal reserve banks, and then order them to apportion these credits out among the banks of the country. This process will certainly lead to an expansion of credits. The former issues of liberty bonds are still carried by the banks to a considerable extent. It may be contended that the bank credit expansion represented by the new victory notes has already occurred in the form of Treasury certificates, which are merely to be funded by the victory notes. The victory note issued thus represents only a shifting of the obligation to pay credits advanced to the Government, a shifting from the shoulders of the banks to the shoulders of the victory note buyers. The volume of outstanding bank credit remains the same. To a certain degree this contention is true. But a portion of the April victory note issue will go to pay future expenditures, not accrued expenditures. Then as soon as the Government needs additional money, it will issue new Treasury certificates, resulting in new extension of bank credit. Moreover, there is little doubt that there will be at least one more Government bond issue during the reconstruction period and this will tend to further increase our present credit structure.

The banks must lend credit and create deposits to meet the expenditures not only of our own Government, but of foreign Governments as well. The same thing results even if these Governments are served directly by private investors here instead of via the United States Treasury. These investors pay for foreign Govern-

ment bonds as they do for our liberty bonds—on the installment plan—paying a small part down and borrowing the rest from the bank. This increased purchasing power will be mostly spent in this country for supplies to be sent abroad for rehabilitation. This continuance of vast loan issues, connected with war and reconstruction throughout the world is a factor which will maintain the high price level temporarily, which means many months.

It is also worth keeping in mind that liberty bonds and other Government securities held here do not wholly cease being a source of credit expansion when the individual subscribers have completed their payments on the bonds and really own them. These new bonds are unrivaled security for further borrowings from banks for commercial purposes, and they will continue to be so until the Government which issues them redeems them.

The availability of the vast issues of war bonds as bases for future credit expansion, coupled with the fact that our banking system has still many unused reefs, sure to be taken out later, when business wishes to spread more sail, is the chief reason why prices will keep up permanently; that is, for many years.

Between the period of temporary and the period of permanent effects, there may be a slight dip in the price level, say a year from now. If so, it is the more incumbent upon business to proceed now; for it can not wait a year.

During the war the flotation of stocks and bonds of commercial concerns has been very greatly diminished. During the period upon which we are now entering the issue of such securities will increase greatly.

Against any considerable reduction in bank credit and hence in the general level of prices, we shall find the whole business community in arms. Falling prices mean hard times for the individual and for the nation and every one resists the tendency. At the end of the Civil War the Treasury started to reduce the quantity of greenbacks. A start had hardly been made, however, before the business depression of 1866 and 1867 caused Congress to forbid by law any further reduction. Should the Federal reserve banks attempt, by raising their discount rate or otherwise, to reduce the volume of bank credit outstanding they will meet with the same sort of opposition. Moreover, the hostile attitude of labor toward the lowering of wages will deter legislators and bankers from any organized policy of contraction.

Looking into the still more remote future, there will be in Europe, particularly on the Continent, a vast increase in deposit banking. The need of the Governments there for funds during war times hastened the introduction of deposit banking. Money went out of circulation into bank vaults, and there became the basis for circulating credits. This means a new habit which will lead to a great currency expansion. Far-away countries, like India and China, are also learning to use deposit banking. It is as if a new source of gold supply had been discovered. What has been discovered is a new way of using the gold supply. The world, during the course of the war, has thus started, or has hastened, an equivalent of the price revolution of the sixteenth century.

Business men should face the facts. To talk reverently of 1913–1914 prices is to speak a dead language to-day. The buyers of the country, since the armistice, have made an unexampled attack upon prices through their waiting attitude, and yet price recessions have been insignificant. The reason is that we are on a new high-priced level, which will be found a stubborn reality. Business men are going to find out that the clever man is not the man who waits, but the one who finds out the new price facts and acts accordingly.

SOCIETY AFFAIRS

NEW YORK SECTION

A meeting, the third and last of the season, was held at the Hotel McAlpin, New York, on February 19, 1919, 61 members and guests being in attendance. The chairman of the Section, W. W. Brush, presided. Charles R. Bettes was elected a member of the Board of Governors for a five-year term. The following papers were presented: "The Chloramine Process as Applied to the Catskill (Esopus) Water," by Dr. Frank E. Hale; "Water Purification Trucks," by Wm. J. Orchard; "Notes on Utility and Performance of Compound Meters," by Fred B. Nelson.

ELEVENTH ANNUAL MEETING OF THE ILLINOIS SECTION

The eleventh annual meeting of the Illinois Section opened at the University of Illinois, Urbana, at 9 a.m., March 25, 1919. All sessions, except the evening meeting, March 25, which was held at Hotel Beardsley, were held in the Engineering Lecture Room of the University. The attendance was 30 members and 31 guests. The following papers were presented:

March 25, morning session. Welcome, by Prof. A. N. Talbot; report of the secretary, by G. C. Habermeyer; report of the treasurer, by H. E. Keeler; "Water Softening," by M. F. Stein; "Lime Softening of Water and the Use of Sludges as an Aid Thereto," by W. A. Sperry; "Treatment of Water of High Silica Content," by O. M. Smith.

March 25, afternoon session. "Financing Purchase of the Quincy Waterworks Plant," by W. R. Gelston; "Typhoid Fever at Moline," by M. C. Sjoblom; "Operation of Moline Filtration Plant," by A. E. Anderson; "Differentia of B. Coli," by Everett Judson; "Stream Pollution," by H. B. Ward; "New Well of Large Capacity at University of Illinois," by M. L. Enger; "Treatment of Water to Prevent Growth of Crenothrix," by F. C. Amsbary; "Chloramine and Crenothrix," by W. F. Monfort.

March 25, evening session. "Letters from the Front," read by W. W. DeBerard; "Porto Rico," (illustrated), by C. B. Burdick; "Water Supply Work in the Advance Section, Service of Supplies, France," by H. E. Babbitt and H. F. Ferguson.

March 26, morning session. "Fatigue of Metals," (motion pictures), by H. F. Moore; "Water Supply Problems of Our State Institutions," by F. J. Postel; "Problems of the State Department of Public Health during the War," by H. P. T. Matte; "Quality of Water in Illinois Streams," by M. E. Jewell; "Unusual Ground Water Supply Conditions at Baton Rouge, La.," by L. R. Howson.

March 26, afternoon session. "The Selection of a Site for an Industrial Housing Development," by P. E. Green; "Old Hickory Powder Works Water Supply," by O. E. Bulkeley; "Testing Stations for Water Supply," by W. T. McClenahan and R. S. Rankin; "Cost of Pumping Through Pipe Lines," by G. C. Habermeyer.

The following resolution was passed:

WHEREAS: Our former President, Captain Paul Hansen, and our former Secretary, Lieutenant Colonel Edward Bartow, and a number of other valued members of this Association, are at present serving or have served with the American Expeditionary Forces,

Be it Resolved by the Illinois Section of the American Water Works Association in convention assembled on this, the twenty-sixth day of March, 1919, that the heartiest commendation and appreciation of their services and sacrifices be extended to them, for upon such unselfish service the world war was won.

The members present at this meeting trust that those who are yet absent, may be permitted to be present at the next session of the Association in full health and strength.

Be it Further Resolved that a copy of this resolution be presented to each member of this Association who thus served or is serving our country, and, also, that it be incorporated in the minutes of proceedings of the Association.

OSCAR E. BULKELEY,
HENRY RINGNESS,
F. C. AMSBARY,
Committee on Resolutions.

Officers for the ensuing year were elected at the morning session on March 26, as follows: Chairman, W. E. Lantz; vice-chairman, F. C. Amsbary; treasurer, H. E. Keeler; trustee, H. M. Ely.

The report of the secretary, G. C. Habermeyer, stated that during the war it was not possible for some members to give much of their time to the affairs of the Section and naturally there had

been a decrease in number of members. The largest membership of this Section was in 1916, when there were 129 active and 14 associate members. The largest membership in the Illinois Water Supply Association was in 1914, when there were 275 active members and 35 associates. Of the active members 162 lived in this state. The list published last September shows 103 active, 3 corporate and 15 associate members. Considering the past, the secretary reported that it should be possible to secure a considerable addition to the membership this year.

The report of the treasurer showed that the Section had no unpaid bills. The expenses of the annual meeting, about $75, could be paid as soon as the bills were received.

BOOK NOTES

FLOW OF WATER THROUGH 1½-INCH PIPE AND VALVES. By Frederick W. Greve, Jr. Engineering Experiment Station, Purdue University, Lafayette, Ind.; paper, 6 x 9 inches, 21 pages.

The results of 270 experiments with 40-foot lengths of commercial black pipe and with gate, globe and check valves, are given. They show larger rates of discharge than those reported by Weisbach, for gate valves.

HYDRAULIC EXPERIMENTS WITH VALVES, ORIFICES, HOSE NOZZLES AND ORIFICE BUCKETS. By Arthur N. Talbot, Fred B. Seely, Virgil R. Fleming, Melvin L. Enger. Engineering Experiment Station, University of Illinois, Urbana, Ill.; 6 x 9 inches, 60 pages, paper, 35 cents.

The first of the four papers in this report described a series of tests on the loss of hydraulic head in 1-inch and 2-inch gate, globe and angle valves. It was found that the loss of head, as the valve is closed from a wide-open position, varies comparatively little until the valve is half closed, after which the loss with globe and gate valves increases rapidly and is considerably larger than that of the angle valves. The second paper gives the results of tests of the flow through small submerged orifices at low velocities. These results show that the coefficient of discharge of circular and square orifices of over 10 square inches area is about 0.6, and that of rectangular orifices is somewhat larger. The third paper gives the results of tests of the discharge of 1½-inch hose and conical nozzles

$\frac{5}{16}$, $\frac{7}{16}$ and $\frac{1}{2}$ inch in diameter, such as are used for interior fire protection. The fourth paper describes the orifice bucket for measuring the discharge from deep well pumps and for similar purposes.

CONVEYANCE AND DISTRIBUTION OF WATER FOR WATER SUPPLY. By Edward Wegmann. New York, D. Van Nostrand Co.; cloth, 6 x 9 inches, 663 pages, $5.00.

This treatise covers aqueducts, pipe lines and distributing systems, and the author hopes to supplement it with another volume on the collection and storage of water.

Part I contains three chapters on the consumption of water, the flow of water in aqueducts, and the flow of water through orifices and pipes. In it he proposes a new formula, devised in collaboration with Albert Ahrens, Jr., for the flow through clean cast-iron pipes. It is $v = 182.5\ r^{0.723}\ s^{0.539}$. It was deduced from a study of 260 experiments, weighted according to the care and accuracy of the investigations.

Part II, on design and construction, describes various types of mains and the stresses arising in water pipes, flexible joints, submerged pipes, gates and valves, hydrants, intakes, aqueducts, service reservoirs, standpipes and tanks, fire protection, high-pressure distribution systems, and distribution systems.

Part III, devoted to maintenance and operation, describes service pipes and connection, cleaning mains and aqueducts, thawing pipes and hydrants, leakage, durability of mains, electrolysis, appliances used by pipe gangs, water waste, pitot tube gaging, water meters, and recording instruments.

There are five appendices, giving specifications for cast iron pipe, hydrants and valves, structural and sheet steel, tables of fire streams and of steel and riveted steel pipe, and the underwriters' requirements for fire streams.

THE RESULTS OF MUNICIPAL ELECTRIC LIGHTING IN MASSACHUSETTS. By Edmond Earle Lincoln, Ph.D. Boston, Houghton Mifflin Co.; cloth, 5 x $7\frac{3}{4}$ inches, 484 pages.

This book gives the results of an investigation of municipal ownership of public utilities. The municipal electric lighting plants of Massachusetts were chosen for the study because the author considered that only in the case of the electric lighting plants of

Massachusetts are there reasonably comparable data of municipal and private plants extending through a period of years. The purpose of the author is to show how future studies of this character may be conducted satisfactorily, "rather than to make any startling contributions to an already overworked field." His first chapter is a survey of the literature of municipal electric lighting in this country, which is criticised freely. The second chapter outlines the Massachusetts laws regarding lighting plants and the work of the Massachusetts Board of Gas and Electric Light Commissioners. The third chapter explains the author's methods of comparing municipal and private lighting.

In the fourth chapter the author takes up in detail the analysis and comparison of physical statistics. He differentiates between generating plants and purchasing plants, the latter buying current from a generating plant. He reports that the physical development of private generating plants has been more active than that of the public plants, but the contrary is true of the physical development of purchasing plants.

Chapters five to eight inclusive discuss financial statistics. Because of unsatisfactory accounting methods, the author reports that entirely comparable figures regarding investment in generating plants are not available; what he gives shows that the actual cash investment is more rapid with private than public plants. The net rates of municipal and private plants are stated to be about the same, although the maximum schedule rates of the companies are higher than those of municipal plants. In a general way, the author concludes that the financial aspects of generating plants furnish "little in the rates and less in the investment policy to cause us to feel optimistic on the subject of municipal ownership in the cases studied." So far as purchasing plants are concerned, there has been far greater economy in investment in the municipal plants than in the companies, and the latter seem to have an unnecessarily high unit operating cost. The rates charged by the municipal plants of this type are considered too low.

In the ninth chapter the author asserts that while wages are the same under private and public ownership, the efficiency of labor is greater under company ownership. The tax rates in cities owning their lighting plants have increased more rapidly than in cities served by private companies. The author states that while cities owning purchasing plants seem content with the service in most cases,

there is considerable dissatisfaction in the case of municipal generating plants. He ascribes much of the success of municipal ownership to the supervision of the Board of Gas and Electric Light Commissioners.

In the tenth to thirteenth chapters, inclusive, the author gives the results of his personal investigations of the plants. Finally, in the fourteenth chapter, he gives his conclusions which he eventually summarizes as follows: "Only the simplest and the well-seasoned enterprises are at all suitable for public operation; and even these are in grave danger of becoming less efficient than they would be in private hands. Though political expediency or social necessity may sometimes momentarily outweigh all economic considerations, it still seems inherent in the nature of things that private industry must continue to show the way."

The book closes with a voluminous bibliography and a large collection of tables. It is one of the series of economic and social studies which were made possible by the Hart, Shaffner & Marx prizes, substantial sums awarded for particularly meritorious monographs of this nature.